Dynamic Response Study on U Shaped Cantilever Retaining Wall

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Abstract- *U shaped cantilever retaining wall includes a set of two parallel vertical walls which are free at the top and fixed at the base and connected to each other through a horizontal slab. This retaining wall can be made more economical and stable by adding a self-weight reduction shelf which helps in reducing the total volume of the backfill soil. Moreover, in circumstances where there is any obstacle near the wall or it is impossible to provide soil reinforcement, these walls can be effective in reducing the cost as well as improving the overall safety and stability of the structure. In this research study, an effort has been made to study the seismic behaviour, stability of the structure, yield acceleration, lateral active earth pressure, dynamic active earth pressure, variation of safe bearing capacity, deflection of the wall under dynamic loading, and natural time period of the retaining wall. In the present study analytical models from different research scholars have been compared with the numerical models created using Wrinkler's Soil Spring in Staad Pro v8i.*

Keywords- yield acceleration, dynamic earth pressure, natural time period

I. INTRODUCTION

Retaining walls are the structures which are used to withstand the Lateral Pressure of earth fill. Retaining wall can be provided at place where there is an abrupt change in the ground levels, such as along high embankment section of highways and railways, basement of high rise buildings etc. Cantilever reinforced concrete retaining walls are the most popular type of retaining walls, since it is easy to construct, install, and considerably economical. In areas where safe bearing capacity (SBC) of soil is very low, it is very difficult to construct the conventional retaining wall for heights above 6m. In such conditions retaining wall with symmetrical shape (like U-Shaped) will help to reduce the moments and stabilize the structure. In order to make the structure more economical, retaining wall with self-weight reduction shelf can also be used. The analysis of such structures involves complex soil structure interaction, which includes the seismic response of structure, dynamic response of the earth fill, flexure response of the structure, dynamic stability of retaining structure. Very less research has been carried out to understand the performance of the retaining walls during earthquakes, moreover retaining wall with pressure shelves, is still a topic of research. In this paper an effort is made to understand the effect of seismic forces, on U shaped cantilever retaining wall with shelves. The analysis is based on comparision of numerical modelling obtained from STAAD Pro v8i program and analytical solutions mentioned by various research scholars.

II. LITERATURE REVIEW

Many researches regarding various aspects of the retaining wall have been carried out by various researchers across the world. Some of these works are briefed below:

Coulomb et.al. [1] proposed a new theory to determine the earth pressure on the retaining walls. This theory was called Coulomb's wedge theory which stated that the earth pressure on the retaining wall would act at a distance of H/3 from the base.

After the Kanto earthquake in 1923, Mononobe and Okabe [6] conducted various shaking table experiments and came up with a new method which was termed as the MO method. This method had certain assumptions which are (i) The backfill material should be dry, cohesion-less, homogenous, isotropic and elastic with a constant internal friction angle and very small or negligible deformation. (ii) The wall should deflect enough to exert full strength along the failure plane. This implies wall rigidity is not considered.

Later, MO method was combined with the Coulomb's wedge theory (with quasi-static inertial force) which resulted in a new equilibrium equation from which the coefficient of active lateral seismic earth pressure was obtained. Hence, the MO method became popular as the extension of the Coulomb's wedge theory and was widely used for new researches as well as the design standards such as Euro code 8 and Australian Standard 4678 to design the retaining walls.

Seed and Whiteman [10] carried out various investigations based on the MO method to know the effect of different factors likes angle of friction, horizontal acceleration, slope of backfill, wall friction, source of load(seismic or blast) on the dynamic earth pressure. They suggested that the dynamic earth pressure could be divided into static and dynamic part. This lead to a simplified version of the MO method and was widely used to solve the issues related to dynamic earth pressure. Moreover, unlike the MO method, Seed and Whiteman proposed the location of the resultant of the force to be a height of 0.6H(H is the height of the wall) from the bottom.

Richard and Elem [8] formulated the serviceability solution(R-E model) with the MO method by using the sliding block model. The R-E model provided a function for gravity wall displacement from which the coefficient of limiting wall acceleration can be calculated. This coefficient could be used in the MO method as the horizontal acceleration to calculate the earth pressure. In 1979, Elms and Richard suggested that Pae and ay can be calculated using the same equation as in MO method.

Newmark et.al., in 1965 [7] found out a procedure for calculating the displacements as well as the time dependent inertial force. This was an extension of the pseudo static approach which suggested that the initiation and movement of the failure slope would occur when the initial forces on the potential sliding mass were reversed. The acceleration on sliding mass was obtained to be excess of yield acceleration and this effective acceleration was obtained by integrating on the time scale. According to Newmark, the earthquake force along horizontal as well as vertical direction act at the centre of gravity of the the retaining wall.

Seed and Makdisi [11] defined the concept of yield acceleration and used it to obtain the displacement. They observed that the displacement occurred along the slide plane when the acceleration of the sliding mass exceeded the yield value. Moreover, they also concluded that permanent acceleration would be generated when the acceleration exceeds the yield value.

Indrajit Chowdhury and Shambhu P. Dasgupta [2] proposed a comprehensive analytical solution on the basis of modal analysis which also took into consideration the effect of time period of the retaining wall. This proved to solve many open issues like c- φ soil, used of logarithmic spiral curves,etc. within the MO framework unlike the pervious methods which could be used for cohesionless soils only. Moreover, this method does not consider the effect of vertical acceleration in the analysis of the retaining walls.

Hany F. Shehata et. al. [5] carried out various parametric studies using PLAX-IS2D-AE.01 to investigate the importance of retaining wall with relief shelves. He concluded that the number of shelves, shelf rigidity and shelf position effected the resultant distribution of lateral earth pressure, top wall movement and maximum flexural moment. He also concluded that on addition of pressure relief shelf, the total active earth pressure reduced. He suggested that for high retaining walls and some special cases (for repair) for constructed walls that have stability problems, cantilever retaining walls with shelves would be recommended.

The objectives of present study are to study the seismic behaviour, dynamic stability of the structure, yield acceleration, lateral active earth pressure, dynamic active earth pressure, variation of safe bearing capacity, deflection of the wall under dynamic loading, and natural time period of the retaining wall

III. METHODOLOGY

The basic problem of the present study was a real life engineering crisis observed during construction of a two Lane highway bridge with an overall width of 12.9m, in the state of Bihar, (India). The site construction site is in seismic zone V and flash flood due to heavy rainfall in the foothill of Himalayas. The safe bearing capacity of the area was relatively very low compared to the neighbouring areas, at the construction site the safe bearing capacity encountered at the depth of 1 m from the ground level was as low as 13t/m2. The highway passed through densely populated areas; hence the right of way restricted the natural flaring of the backfill soil, thereby proposing a requirement for retaining wall of height 11 m, in order to restrain the backfill soil from spreading. The tabulated form of the problem statement is stated below in table 3.1.

TABLE 3.1: PROBLEM STATEMENT PARAMETERS

Seismic Zone	:	V
Overall width of the bridge	:	12.9m
Overall height of the retaining wall	:	11.0m
Angle of internal friction of the backfill soil 'Φ1'	:	30°
Cohesion of the backfill soil 'c1'	:	0
Unit weight of the backfill soil 'γ1'	:	18 KN/m^3
Angle of internal friction of the founding soil 'Φ2'	:	35°
Cohesion of the founding soil 'c2'	:	0
Unit weight of the founding soil 'γ2'	:	20 KN/m^3
Safe bearing capacity at a depth of 1m below OGL, 'SBC'	:	13 t/m^2

3.1 PSEUDO-STATIC ANALYSIS

This is one of the most commonly used method of seismic analysis for embankment and slopes. It is carried out by introducing a permanent pseudo-static body force which represents the earthquake forces.

$F_h = ma_h$ ……….……………………………..(3.1)

$F_v = ma_v$ …...……………………………..(3.2)

where, ah & av are horizontal and vertical pseudo static acceleration due to seismic effect respectively.

$F_h = \frac{w}{g} a_h = k_h w$ ……………………………………………(3.3)

$F_v = \frac{w}{g} a_v = k_v w$ ……………………..(3.4)

where, kh and kv are pseudo static seismic coefficient along horizontal and vertical direction respectively.

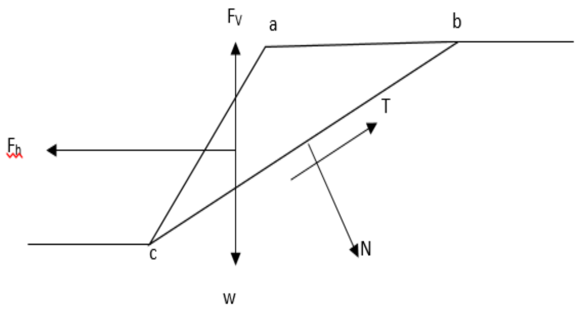

FIGURE 3.1 SOIL MASS AFTER FAILURE

Where,

bc	:-	Surface of failure
T	:-	Shear Force
N	:-	Normal Reaction Force
W	:-	Weight of the failure Surface
Fh	:-	Pseudo static (Seismic) Force in horizontal direction
Fv	:-	Pseudo static (Seismic) Force in vertical direction

3.2 ANALYSIS BASED ON MONONOBE AND OKABE THEORY

Coulomb[1] (1776) developed the wedge theory to determine the lateral earth pressure on the retaining wall. Mononobe[6](1929) figured out the method to find the dynamic earth pressure on the retaining walls, which are stated below:-

$P_a = \frac{1}{2} k_a \gamma_s H^2$ ………...…………………………………(3.5)

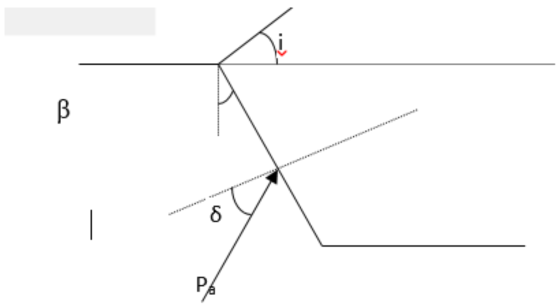

FIGURE 3.2 RETAINING WALL UNDER STATIC EARTH PRESSURE

$$ka = \frac{\cos^2(\Phi-\beta)}{\cos^2\beta\cos(\delta+\beta)\left[1+\left(\frac{\sin(\delta+\Phi)\sin(\Phi-i)}{\cos(\delta+\beta)\cos(\beta-i)}\right)^{0.5}\right]^2} \quad \text{……………….(3.6)}$$

ka	:-	Coefficient of active earth pressure
	:-	Unit weight of soil in kN/m3
H	:-	Height of Earth fill Retaining Wall

Under Seismic Condition

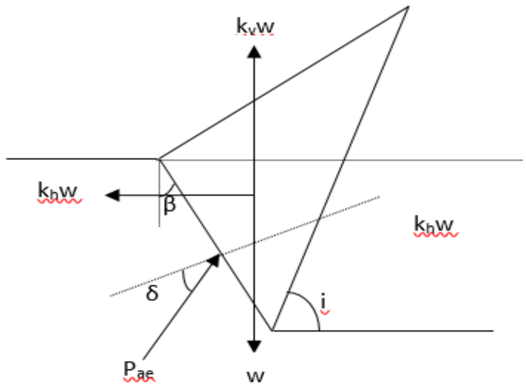

FIGURE 3.3 RETAINING WALL UNDER DYNAMIC EARTH PRESSURE

$$P_{ae} = \frac{1}{2}\gamma_s H^2 k_{ae}(1-k_v) \quad \text{………………………(3.7)}$$

$$kae = \frac{\cos^2(\Phi-\theta-\beta)}{\cos\theta\cos^2\beta\cos(\delta+\beta+\theta)\left[1+\left(\frac{\sin(\delta+\Phi)\sin(\Phi-\theta-i)}{\cos(\delta+\beta+\theta)\cos(\beta-i)}\right)^{0.5}\right]^2} \quad \text{………………(3.8)}$$

$$\theta = \tan^{-1}\left(\frac{k_h}{1-k_v}\right) \quad \text{………………………(3.9)}$$

$\Phi - \theta - i < 0$

If	$\Phi - \theta - i < 0$	Then no real solution is possible for k_{ae}	
	$i \le \Phi - \theta$	For real solution	
If	$\theta = 0$	Stability under No seismic Condition	$i \le \Phi$
If	$i = 0$	Seismic Condition for stability	$\theta \le \Phi$

Then no real solution is possible for k_{ae}

$\tan\theta = \frac{k_h}{1-k_v}$

$\tan\theta \le \tan\Phi$

$k_h \leq (1 - k_v) \tan \Phi$

……………………………………………………..…..….(3.10)

$k_{h,critical} \leq (1 - k_v) \tan \Phi$

…..……………………………………………..…..….(3.11)

Point of Application

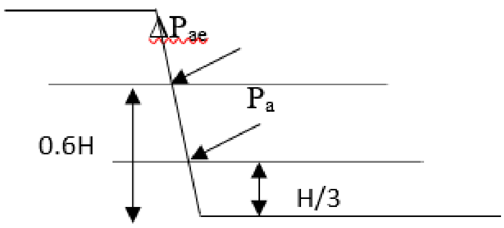

FIGURE 3.4 POINT OF APPLICATION FOR STATIC AND DYNAMIC EARTH PRESSURE

$\Delta P_{ae} = P_{ae} - P_a$ ……………………..…….…....…..….(3.12)

$\bar{H} = \frac{P_a \frac{H}{3} + \Delta P_{ae} \frac{3H}{5}}{P_{ae}}$ …………………….……………..….....….(3.13)

3.3 YIELD ACCELERATION

Newmark[7] in 1965 gave a procedure for calculation of displacements in association with the time dependent inertial force which was an extension for the pseudo-static approach.

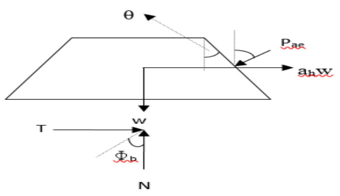

FIGURE 3.5. RETAINING WALL SHOWING ALL FORCES

$N = w + P_{ae} \sin(\delta + \theta)$ ……….………….….(3.14)

$T = a_h w + P_{ae} \cos(\delta + \theta)$ …..…….…………..….(3.15)

$T = N \cos \Phi_b$ ...……………………………..….….(3.16)

Pae :- Dynamic lateral force during seismic loading (MO Method)

Φb :- Friction angle between base and founding soil

Seed and Makdisi[10] defined yield acceleration and used this concept to obtain displacement. Displacement will occur along the slide plane when the acceleration of sliding mass exceeds the yield value. When the acceleration exceeds yield value permanent acceleration is generated.

$a_h = \left[\tan \Phi_b - \frac{P_{ae}\cos(\delta+\theta) - P_{ae}\sin(\delta+\theta)\tan \Phi_b}{w}\right]$ (3.17)

$a_h = \frac{a_y}{g}$ ……………...……...…..……………….....….(3.18)

$a_y = \left[\tan \Phi_b - \frac{P_{ae}\cos(\delta+\theta) - P_{ae}\sin(\delta+\theta)\tan \Phi_b}{w}\right] g$……………………… ……………….…………...…….(3.19)

ay :- yield acceleration

3.4 DISPLACEMENT ANALYSIS

Elms and Richard[8] in 1979 suggested that Pae can be calculated using MO Method, ay can be calculated using equation 3.19. They came up with the following expression to find out permanent displacement.

$d_{perm} = 0.087 \frac{v_{max}^2 a_{max}^3}{a_y^4}$ …………………………….….(3.20)

vmax	:-	peak ground velocity
amax	:-	peak ground acceleration
ay	:-	yield acceleration

3.5 DYNAMIC STABILITY ANALYSIS

According to Newmark's theory[7], the failure slope would be initiated and movement would occur when the initial forces on potential sliding mass are reversed. Newmark calculated acceleration on sliding mass is to obtain in excess of yield acceleration. From this effective acceleration, velocity and displacement are obtained when integrating on time scale. The earthquake force acts along two perpendicular directions the horizontal and the vertical direction acting at the centre of gravity of the retaining wall.

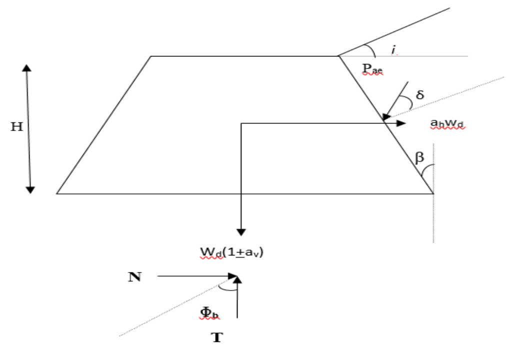

FIGURE 3.6 RETAINING WALL UNDER DYNAMIC EARTH PRESSURE

Under Dynamic Condition

N = Sum of forces in vertical direction

T = Sum of forces in horizontal direction

For sliding at base

$T = N \tan \Phi_b$ ………………….……….….(3.21)

Along vertical direction

$N = w_d \pm a_v w_d + P_{ae} \sin(\beta + \delta)$ ……...….(3.22)

Solving,

$w_d = P_{ae} \left[\frac{\cos(\beta+\delta)-\sin(\beta+\delta)\tan\Phi_b}{(1+a_v)\tan\Phi_b - a_h}\right]$…..…..….(3.23)

Substituting the value of Pae and ah = (1 + av) tanΨ

$w_d = \frac{1}{2}\gamma_s H^2 k_{ae}\lambda_d$ ……….……………......….(3.24)

$\lambda_d = \frac{\cos(\beta+\delta)-\sin(\beta+\delta)\tan\Phi_b}{(1\pm a_v)(\tan\Phi_b - \tan\Psi)}$………………..…...(3.25)

Under static condition

Weight of retaining wall

$w_s = \frac{1}{2}\gamma_s H^2 k_a \lambda_s$ ……………………......…..(3.26)

$\lambda_s = \frac{\cos(\beta+\delta)-\sin(\beta+\delta)\tan\Phi_b}{\tan\Phi_b}$

……….……………………….....…..(3.27)

$\frac{w_d}{w_s} = \frac{k_{ae}}{k_a} = \frac{\tan\Phi_b}{(1\pm a_v)(\tan\Phi_b - \tan\Psi)}$

……………………….……….....………....(3.28)

$F_T = \frac{k_{ae}}{k_a}$………………………......…...…….(3.29)

$F_1 = \frac{\tan\Phi_b}{(1\pm a_v)(\tan\Phi_b - \tan\Psi)}$ ……………….........(3.30)

$\frac{w_d}{w_s} = F_T F_1 = F_w$ …….....……..…………...…...(3.31)

Fw :- Factor of safety for dynamic stability

3.6 DYNAMIC RESPONSE OF DRY COHENSIONLESS BACKFILL

S.P.Dasgupta and I. Chowdhury[2] proposed an analytical solution with modal analysis which takes time period of the wall into consideration, which were mostly ignored by previous researchers. They can up with the most practical approach for the solution of the most generalised cases. They assumed that the soil section under active case will be at incipent failure when the line of failure makes an angle equal tan(450+Φ/2), since the already failed soil mass under static condition will not give any stiffness to the dynamic response but will only be contributing to the inertial effect. The cantilever lever wall mass was ignored and only contributed to the stiffness for the soil-structure system. The retaining wall was considered fixed at the base and foundation commpliances were ignored for the analysis.

$k_a = \frac{1-\sin\theta}{1+\sin\theta}$ ………….…………………………..(3.32)

$P_a = k_a \gamma_s z$ ……………………………………….(3.33)

The walls are considered as cantilever member fixed at base of the foundation and hence the differentiating equation of static equilibrium.

$EI\frac{d^4u}{dz^4} = k_a\gamma_s z$ …………...…………………….(3.34)

u :- displacement of wall

E :- Young's Modulus of the wall

I :- Moment of Inertia of the wall

On Integrating successively

$EI\frac{d^3u}{dz^3} = \frac{k_a\gamma_s z^2}{2} + c_1$ …………….…………..(3.35)

$EI\frac{d^2u}{dz^2} = \frac{k_a\gamma_s z^3}{6} + c_1 z + c_2$ ……………………..(3.36)

$EI\frac{du}{dz} = \frac{k_a\gamma_s z^4}{24} + \frac{c_1 z^2}{2} + c_2 z + c_3$ ………………(3.37)

$EIu = \frac{k_a\gamma_s z^5}{120} + \frac{c_1 z^3}{6} + \frac{c_2 z^2}{2} + c_3 z + c_4$ ……….…..(3.38)

For the given wall

$z = 0, \quad \frac{d^3u}{dz^3} = 0, \quad c_1 = 0$

$z = 0, \quad \frac{d^2u}{dz^2} = 0, \quad c_2 = 0$

$z = H, \quad \frac{du}{dz} = 0, \quad c_3 = -\frac{k_a\gamma_s H^4}{24}$

$z = H, \quad u = 0, \quad c_4 = \frac{k_a\gamma_s H^4}{30}$

$$EIu = \frac{k_a\gamma_s z^5}{120} - \frac{k_a\gamma_s H^4 z}{24} + \frac{k_a\gamma_s H^5}{30}$$

Now the Equation 3.38 can be rewritten as

…………..……………………….………….………...(3.39)

Equation 3.39 can be generalised as

$u = \frac{k_a\gamma_s H^5}{30}\left[\frac{\xi^5}{4} - \frac{5\xi}{4} + 1\right]$, where $\xi = \frac{z}{H}$ ………..(3.40)

$u_{static} = \frac{k_a\gamma_s H^5}{30}$ at $\xi = 0$ …………..…………..(3.41)

The natural time period of the wall

$T = 2\pi\sqrt{\frac{u_{static}}{g}}$ ……………...………..………...(3.42)

When t is the thickness of the wall

$T_a = 3.97\sqrt{\frac{k_a\gamma_s H^5}{Et^3 g}}$ ……………………..….…..(3.43)

$s_d = \frac{Sa}{w^2}$ as (Clough 1984)…………………..(3.44)

Sa :- Spectral Acceleration

$\omega = \frac{2\pi}{T}$, natural frequency of the wall

In the terms of code equation 3.44 can be expressed as

$s_d = \kappa B \frac{Sa}{w^2}$ ……………………………….……..(3.45)

κ :- mass model participation factor &

$u = \kappa B \frac{S_a}{4\pi} T^2 f(\xi)$……………………..….…. (3.46)

Where,

$f(\xi) = \frac{\xi^5}{4} - \frac{5\xi}{4} + 1$ …………….…………. (3.47)

$u = \kappa B \frac{k_a \gamma_s H^5}{30} \left(\frac{S_a}{g}\right) \left[\frac{\xi^5}{4} - \frac{5\xi}{4} + 1\right]$ ….……… (3.48)

Considering,

$M = EI \frac{d^2u}{dz^2}$, $V = EI \frac{d^3u}{dz^3}$

Then

$M_\xi = \kappa B \frac{k_a \gamma_s H^3}{6} \left(\frac{S_a}{g}\right) [\xi^3]$……………….……………………………….. (3.49)

$V_z = \kappa B \frac{k_a \gamma_s z^2}{2} \left(\frac{S_a}{g}\right)$……………………………………………….. (3.50)

Equation 3.48, 3.49, 3.50 are exact and gives the dynamic displacement, moment and shear for a cantilever retaining wall under seismic force in fundamental mode for cohesion less dry back fill.

The modal participation factor **κ** can be expressed as

$\kappa = \frac{\int_0^1 (\gamma_s H^2 \xi) f(\xi) d\xi}{\int_0^1 (\gamma_s H^2 \xi) f(\xi)^2 d\xi}$……………………………………………. (3.51)

$\kappa = \frac{\int_0^1 \xi \left(\frac{\xi^5}{4} - \frac{5\xi}{4} + 1\right) d\xi}{\int_0^1 \xi \left(\frac{\xi^5}{4} - \frac{5\xi}{4} + 1\right)^2 d\xi}$……………………………………………. (3.52)

Equation 3.52 can be numerically solved using numerical method.

Effect of vertical acceleration Sv

The equation 3.34 can be concluded

$P = EI \frac{d^4u}{dz^4} = k_a \gamma_s z$……………………………………………….. (3.53)

Dynamic Pressure on the wall can be expressed as

$P_{dynamic} = EI \frac{d^4u}{dz^4} = \kappa B k_a \gamma_s \left(\frac{S_a}{g}\right) z$ ……………………………………………. (3.54)

Sv is the vertical acceleration corresponding to time period Ta, then the dynamic pressure in vertical direction can be expressed as

$P_{v(dynamic)} = \pm \kappa B \gamma_s \left(\frac{S_v}{g}\right) z$ ……………………………………………. (3.55)

The effect of pressure in the horizontal direction can be expressed as

$P_{H(dynamic)} = \pm \kappa B k_a \gamma_s \left(\frac{S_v}{g}\right) z$……………………………………….. (3.56)

The total dynamic pressure when considering the vertical component of acceleration are expressed as

$P_{dynamic} = \kappa B k_a \gamma_s \left(\frac{S_a}{g}\right) \pm \kappa B k_a \gamma_s \left(\frac{S_v}{g}\right) z$ ……………………………………. (3.57)

When we consider only the positive sign then we can get the maximum pressure

$P_{dynamic} = \kappa B k_a \gamma_s \left(\frac{S_a}{g}\right) + \kappa B k_a \gamma_s \left(\frac{S_v}{g}\right) z$ ……………………………….. (3.58)

From IS 1893:2016 Sv = Sa/2, then equation 3.58 can be expressed as

$P_{dynamic} = \kappa B k_a \gamma_s \left(\frac{3S_a}{2g}\right) z$ ………………………………………….. (3.59)

The dynamic displacement, moment and shear considering the effect of vertical acceleration as

$u = \kappa B \frac{k_a \gamma_s H^5}{20EI} \left(\frac{S_a}{g}\right) \left[\frac{\xi^5}{4} - \frac{5\xi}{4} + 1\right]$ ……………………………………. (3.60)

$M_z = \kappa B \frac{k_a \gamma_s z^3}{4} \left(\frac{S_a}{g}\right)$ ……… ……..… ……..… …….. …………………………..……….. (3.61)

3.7 NUMERICAL MODELLING

Numerical model for the above models were created using STAAD Pro V8i program using line element. Wrinkler's Soil spring[4] was considered for modelling of foundation and soil structure interaction. He idealized soil medium as a system of similar mutually independent closely spaced, discrete, linear elastic spring. According to this idealisation, deformation of foundation due to load applied are confined to loading region only. The stiffness of elastic spring were calculated using equation 3.62 and 3.63. The line elements were loaded with dead load, static earth pressure, dynamic earth pressure, seismic force along x direction. The spacing between two consecutive spring were kept to be 0.30 m, since we have taken unit width into account the area of the spring will be 0.30 m2.

$k_i = k_S A$………………………………………………………….. ………………. (3.62)

$k_S = \frac{QF}{\delta_s}$……………………………………………………………… …..……………(3.63)

Where

ki	:-	Soil spring constant
ks	:-	Modulus of subgrade Reaction
A	:-	Spacing of spring
Q	:-	Base pressure
F	:-	Factor of Safety
δs	:-	Allowed Settlement for foundation

3.8 RESEARCH MODEL DETAILS

An approach was made to find the seismic response of the retaining wall based on analytical solution and numerical modelling. The analytical solutions were performed using Excel Spread sheet and numerical models were created using 1-D line element in STAAD Pro v8i program. The results obtained from different analytical solutions and numerical model were compared. A total of 10 models were developed for the purpose of study. The numerical model used soil spring for SSI and for analytical study the entire system's weight was considered. The walls were considered fixed to the bottom slab and free at the top. The models were studied for bending moment of the wall, deflection of the wall, effectiveness of the self-weight reduction shelf, yield acceleration of the wall and the static and dynamic earth pressure. The geometry provides an advantage from the overturning moments by balancing the moments are the centre of base slab. The dynamic stability for these retaining walls were verified by conventional analytical solution.

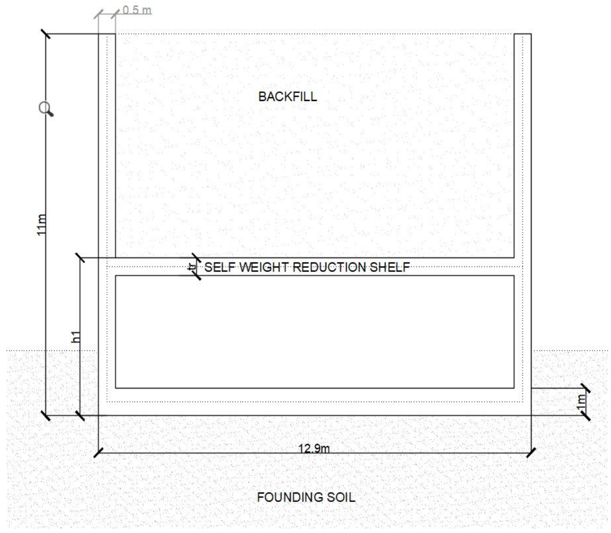

FIGURE 3.7 DISCRETE MODEL OF STUDY

TABLE 3.2 DETAILS OF MODEL

MODEL NO	tr (m)	h_1 (m)
M1	-	-
M2	1	3
M3	1	4
M4	1	5
M5	1	6
M6	1	7
M7	1	8
M8	1	9
M9	1	`10

IV. RESULTS AND DISCUSSION

The analyses of models M1 to M9 were performed through both analytical and numerical modelling. The results of these models were compared with each other. The table 4.1 to table 4.3 shows the variation of earth pressure, deflection, bending moment on the wall, horizontal earthquake force, yield acceleration, soil spring constant, natural time period, factor of safety for dynamic stability, and the safe bearing capacity with respect to the position of shelf from the bottom of the base slab.

TABLE 4.1 VARIATION OF STATIC AND DYNAMIC EARTH PRESSURE

MODEL NO	STATIC EARTH PRESSURE (kN/Sqm)	DYNAMIC EARTH PRESSURE (kN/Sqm)		DEFLECTION (mm)	
	COULOMB'S THEORY	MONONOBE & OKABE	DASGUPTA & CHOWDHURY	ELMS AND RICHARD	DASGUPTA & CHOWDHURY
M1	403.581	406.795	126.861	42.00	25.00
M2	213.464	215.164	101.596	23.00	11.00
M3	163.433	164.735	87.183	21.00	7.00
M4	120.074	121.030	70.897	19.00	4.00
M5	83.384	84.048	55.479	17.00	2.00
M6	53.366	53.791	56.080	16.00	1.00
M7	30.018	30.257	30.964	15.00	0.30
M8	13.342	13.448	21.044	14.00	0.10
M9	3.335	3.362	12.971	13.00	0.03

Table 4.1 shows the variation of Static and dynamic earth pressure from M1 to M9, which is due to the decrease in the height of backfill in each of the models. The next observation is the variation of dynamic earth pressure, in the two methods of calculation, in MO method the earth pressure is calculated on the basis of soil properties and not on the basis of dynamic nature of soil, whereas in the method followed by Dasgupta and Chowdhury the value of coefficient of horizontal acceleration is considered. Deflection is also seen to vary from M1 to M9, which is due to the decrease in backfill, though the values of both the methods are significantly different from each other, in Elms and Richard's calculation the zone factor, and acceleration coefficient are not taken into account which is taken care in Method followed by Dasgupta and Chowdhury.

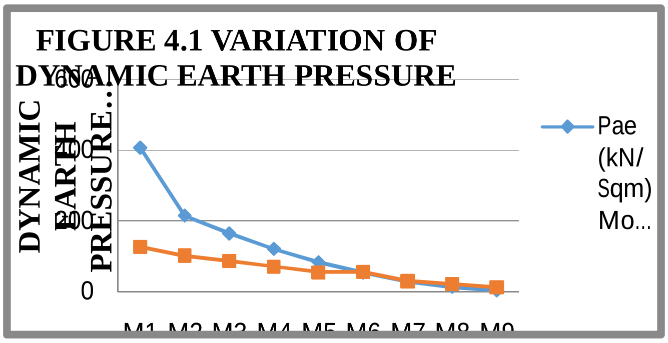

FIGURE 4.1 VARIATION OF DYNAMIC EARTH PRESSURE

From figure 4.1 it is clear that dynamic earth pressure significantly decrease from M1 to M9, and the values of dynamic earth pressure for MO Method is greater than Dasgupta and Chowdhury's method from M1 to M5 then almost equal for M6 and M7 and then for M8 and M9 the value obtained from MO Method were lower when compared to Dasgupta and Chowdhury. Dasgupta and Chowdhury have considered The Zone Factor, Importance Factor, Response Reduction Factor and Response Spectra for the calculation of dynamic pressure on the wall.

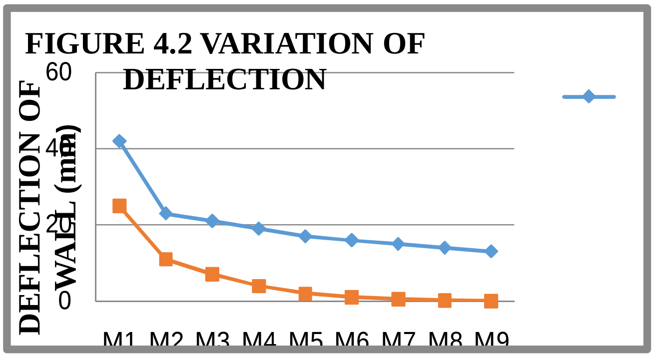

FIGURE 4.2 VARIATION OF DEFLECTION

From figure 4.2 it is clear that deflection significantly decrease from M1 to M9, and the values of Elms and Richard's method is higher than Dasgupta and Chowdhury's method. Dasgupta and Chowdhury have considered the Importance Factor, Response Spectra, Response Reduction Factor and Zone Factor for the calculation of dynamic pressure on the wall.

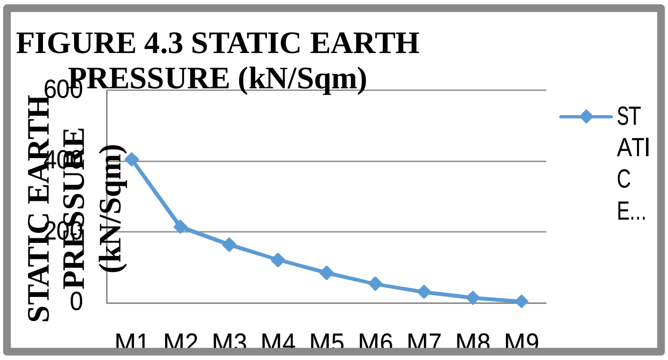

FIGURE 4.3 STATIC EARTH PRESSURE (kN/Sqm)

From figure 4.3 it is understood that static earth pressure significantly decrease from M1 to M9. The reduction in height of the backfill reduces the lateral static earth pressure.

TABLE 4.2 VARIATION OF BENDING MOMENT, FACTOR OF SAFETY FOR DYNAMIC STABILITY, YIELD ACCELERATION

MODEL NO	BENDING MOMENT (kNm)			FACTOR OF SAFETY FOR DYNAMIC STABILITY	YIELD ACCELERATION (m/s^2)
	COULOMB THEORY	DASGUPTA & CHOWDHURY	STAAD PRO v8i		
M1	1500.870	2558.000	1503.735	2.865	4.229
M2	577.310	1371.550	579.682	2.372	4.926
M3	386.780	929.956	389.157	2.377	5.045
M4	243.570	578.990	245.947	2.377	5.162
M5	140.960	332.873	143.337	2.377	5.274
M6	72.167	233.671	74.544	2.377	5.381
M7	30.450	82.569	32.827	2.377	5.479
M8	9.021	31.566	11.398	2.377	5.566
M9	1.128	8.649	3.505	2.377	5.632

From Table 4.2 shows that the bending moment obtained from Coulomb Theory, and Staad Pro V8i are almost equal, and the bending moment obtained from Dasgupta and Chowdhury's Method is higher than the values found from Coulomb's theory and Staad Pro V8i results. This variation amongst results are due to the consideration of seismic parameters (Z, I, R, Sa/g) by Dasgupta and Chowdhury. The Magnitude of Bending moment decrease from Model M1 to M9. Factor of safety for dynamic stability is almost equal for all models with shelf. The yield acceleration increases from Model M1 to M9.

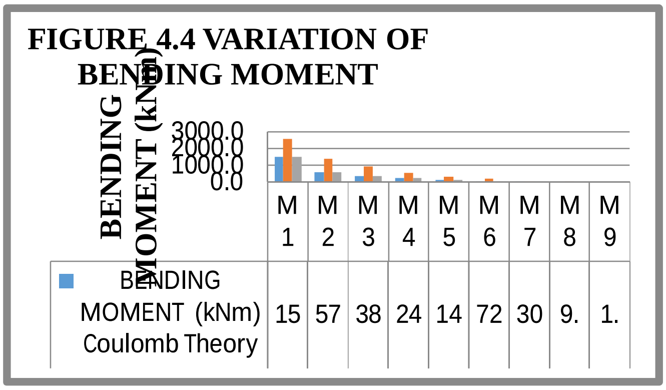

FIGURE 4.4 VARIATION OF BENDING MOMENT

From Figure 4.4 it is pertinent to mention that the bending moment obtained from Coulomb Theory, and Staad Pro V8i are almost equal, and the bending moment obtained from Dasgupta and Chowdhury's Method is higher than the values found from Coulomb's theory and Staad Pro V8i results. This variation amongst results are due to the consideration of seismic parameters (Z, I, R, Sa/g) by Dasgupta and Chowdhury.The Magnitude of Bending moment decrease from Model M1 to M9.

TABLE 4.3 VARIATION OF NATURAL TIME PERIOD, SOIL SPRING CONSTANT, HORIZONTAL EARTHQUAKE FORCE AND SAFE BEARING CAPACITY

MODEL NO	NATURAL TIME PERIOD (s)	SOIL SPRING CONSTANT Ki (kN/m)	HORIZONTAL EARTHQUAKE FORCE (STAAD PRO v8i) (kN)	SAFE BEARING CAPACITY (kN/Sqm)
M1	0.045	6866	178.5	229
M2	0.049	6451	369.4	215
M3	0.050	5898	69.2	197
M4	0.051	5344	26.7	178
M5	0.053	4791	20.2	160

M6	0.056	4237	6.5	141
M7	0.060	3684	3.8	123
M8	0.068	3130	46.7	104
M9	0.087	2577	9.3	86

From Table 4.3 it is clear that the natural time period for the retaining wall increases from M1 to M9 and the Soil Spring constant, and the safe bearing capacity decreases from M1 to M9. The reduction in Soil spring constant and SBC are due to the reduction of the self-weight of the retaining wall. The increment in natural time period is due to reduction in the stiffness of the retaining wall, whereas the horizontal earth quake force is maximum for the M2 and minimum for M7.

V. CONCLUSION

In general the dynamic earth pressure, bending moment and deflection obtained from different analytical solutions and numerical modelling differ significantly from each other. This is due to the consideration of seismic parameters like Zone factor, Importance factor, Response Reduction Factor, and Response Spectra by Dasgupta and Chowdhury in their research. It is also pertinent to mention that M9 is the safest and most economical section for construction as far as analytical solutions are considered but the numerical modelling suggest that M7 should be considered for construction as the net horizontal seismic force is minimum and bending moment of the part of wall below the shelf is relatively lower than M9. The study suggests that the position of shelf should be 0.91H above the base of the founding slab for the safest and most economical construction of retaining wall as far as analytical solutions are considered. However, the numerical modelling suggests that the position of shelf should be 0.73H above the base of the founding slab to be considered for construction of retaining wall. This is because the net horizontal seismic force is minimum and bending moment of the part of wall below the shelf is relatively lower. Hence it can be concluded that the economical shelf positon in the retaining wall was computed to be between 0.73H to 0.91H above the base of the founding slab. It can also be concluded from the study that the self-weight reduction shelf decreases the self-weight of the overall structure and thereby reducing the volume of earth fill. As we all know that the backfill material used for construction in India is limited and hence this design provides a sustainable solution for construction of retaining wall in Indian highway without reducing its usability.

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