Effects of Impulsive Loading on Reinforced Concrete Structures

Saeed Ahmad[1], Mehwish Taseer[2], Huma Pervaiz[3]

ABSTRACT

This paper presents the summary of the work done in civil engineering department, UET Taxila to evaluate reinforced structural elements under impulsive loading. Part of this work has been published elsewhere separately. The experimental work was carried out in the suburbs of Hassan Abdal hills by constructing different wall panels. In this paper a set of four reinforced concrete walls with varying thickness were constructed. These walls were tested with varying explosive loads and scaled distance. Pressure sensors, accelerometers, dynamic strain amplifier, data acquisition board and strain gauges were used to measure air blast and ground shock parameters. Acceleration and the pressure time history at different points are recorded. Empirical relationships have been developed for various blasts loading parameters are compared with the results of previous researchers. In the light of TM 5-855 comparison is made on the basis of air blast and ground shock parameters with the results obtained from Conwep. Detailed analysis of walls subjected to blast loads is carried out using Sap 2000 (Ver. 14). Reinforced concrete wall was designed for blast loading using methodology described in UFC_3_340_02. It is concluded that consideration of air blast and ground shock pressure are important for accurate analysis of structure response of structures.

Keywords: Scaled distance, ground shock, the pressure time history, impulsive loading

Introduction

In the past few decades considerable emphasis has been given to problems of impulsive loading e.g., blast loads and earthquakes. Due to different accidental or intentional events, the behavior of structural components subjected to impulsive loads specifically blast loading has been the subject of considerable research effort in recent years.

Conventional structures, particularly that above grade, normally are not designed to resist impulsive loads; and because the magnitudes of design loads are significantly lower than those produced by most explosions, conventional structures are susceptible to damage from explosions. With this in mind, developers, architects and engineers increasingly are seeking solutions for potential blast situations, to protect building occupants and the structures.

In the design of protective structures to resist the effects of accidental explosions, the principal effects of the explosive output to be considered are blast overpressures referred to as blast pressures. Fragments generated by the explosion and the shock loads produced by the shock wave transmitted through the air or ground. Of these three parameters, the blast pressures are usually the governing factors in determination of the structure response. However, in some situations, fragments and/or shock loads may be just as important as the pressures in determining the configuration of the facility.

The impulsive effects of an explosion are in the form of a shock wave composed of a high intensity shock front which expands outward from the surface of the explosive into the surrounding air. As the wave expands, it decays in strength, lengthens in duration, and decreases in velocity. This phenomenon is caused by spherical divergence as well as by the fact that the chemical reaction is completed, except for some afterburning associated with the hot explosion products mixing with the surrounding atmosphere.

As the wave expands in air, the front impinges on structures located within its path and then the entire structure is engulfed by the shock pressures. The magnitude and distribution of the blast loads on the structure arising from these pressures are a function of the following factors: (1) explosive properties, namely type of explosive material, energy output (high or low order detonation), and weight of explosive; (2) the location of the detonation relative to the protective structures; and (3) the magnitude and reinforcement of the pressure by its interaction with the ground barrier, or the structure itself.

[1-3] Department of Civil Engineering, University of Engineering and Technology Taxila

Experimental Setup

Experimental data is shown in tabular form below:

Table 1: Experimental Data

Wall	Geometry			Stand-off Distance (m)	Charge Weight (kg)	TNT Equivalent (kg)	Scaled Distance ($m/kg^{1/3}$)
	Thickness (in)	Width (m)	Clear Height (m)				
Wall 1	5	2	2	2.87	1	0.6	3.4
				2.87	2	1.2	2.7
				2.87	4	2.4	2.1
Wall 2	6.7	2	2	2.87	1	0.6	3.4
				2.87	2	1.2	2.7
				2.87	4	2.4	2.1
Wall 3	6.4	2	2	2.87	1	0.6	3.4
				2.87	2	1.2	2.7
				2.87	4	2.4	2.1
Wall 4	6.4	2	2	2.87	1	0.6	3.4
				2.87	2	1.2	2.7
				2.87	4	2.4	2.1

Location of the Sensor and Accelerometer for measuring pressure and acceleration respectively are shown in below sketch.

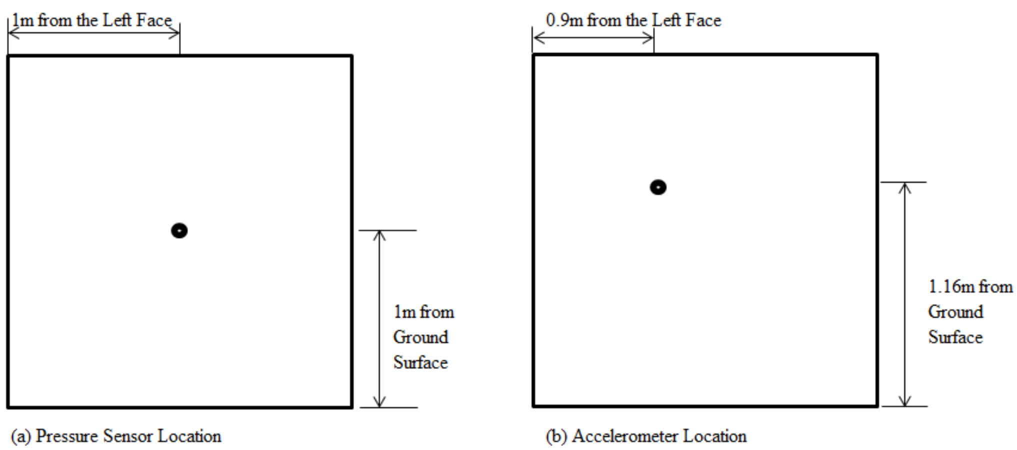

Figure 1: Location of Gauge

Analytical Approaches

SAP2000 is used for detailed analytical analysis of RCC walls under impulsive loading. Input to SAP was extracted from the experimental data. Stresses and moments are noted and shown in below tables.

Table 2: Stress Summary at Wall

Explosive Load	S11 (psi)	S22 (psi)	S12 (psi)
1kg	75.78	140.07	50.377
2kg	152.42	281.73	101.32
4kg	487.26	923.87	337.38

Table 3: Moment Summary at Wall

Explosive Load	M11 (kip-ft/ft)	M22 (kip-ft/ft)	M12 (kip-ft/ft)
1kg	2.77	5.186	1.744
2kg	5.572	10.43	3.51
4kg	7.94	14.85	11.68

ConWep is used for calculating the peak free-field stress due to the directly transmitted shock wave, and optionally allows the addition of a reflected wave from a deeper layer and a relief (tension) wave reflected from the ground surface. Peak particle velocity, acceleration, and displacement are calculated using the direct path only-- reflections from the surface or a lower layer are not included. The output summary of analysis performed using ConWep is shown in below table.

Table 4: Output Summary of ConWep Analysis

TNT Equivalent Weight (kg)	Range (m)	Scaled Distance ($m/kg^{1/3}$)	P_{so} (kPa)	PPA (m/s2)
0.6	2.87	3.40	77.87	134.2
1.2	2.87	2.70	135.6	233.6
2.4	2.87	2.14	236.1	406.7

Results and Discussions

Peak Pressure

Experiments have yielded the following equation for P_{so} with respect to the scaled distance.

$$P_{so} = 1595.6 \left(\frac{R}{Q^{1/3}}\right)^{-2.868}$$

Where P_{so} is in kPa, R is in meters and Q is TNT equivalent charge weight in kg. This relationship is valid for small stand-off distances i.e., R ≤ 5m. This equation has been compared with other empirical formulae as mentioned in below table.

Table 5: Comparison of Empirical Equations for P_{so}

TNT Weight (kg)	Range (m)	Scaled Distance ($m/kg^{1/3}$)	P_{so}					
			Brode Eq. (kPa)	Henrych Eq. (kPa)	Chengqing Eq. (kPa)	Javed Iqbal Eq. (kPa)	Adeel Eq. (kPa)	Current Research (kPa)
0.6	2.87	3.4	53.8	62.9	86.2	97.9	97.9	47.7
1.2	2.87	2.7	83.8	97.2	136.5	153.1	197.2	92.4
2.4	2.87	2.14	135.1	151.68	217.8	237.8	326.8	180

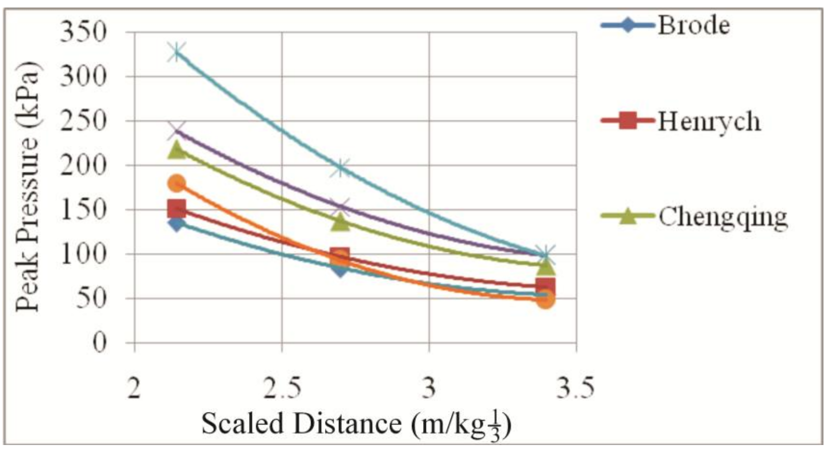

Figure 2: Graph Showing Comparison of Empirical Equations for P_{so}

It shows that the equation derived through experiments is in close relation with Brode's and Henrych's empirical results. Similarly the experimental results are compared with the results obtained from ConWep analysis.

Table 6: Comparison of ConWep and Experimental Output for Pso

Explosive Weight (kg)	Range (m)	Scaled Distance ($m/kg^{1/3}$)	P_{so} (kPa)	
			ConWep	Experimental
0.6	2.87	3.40	77.9	71.7
1.2	2.87	2.70	135.5	137.8
2.4	2.87	2.14	236	234.4

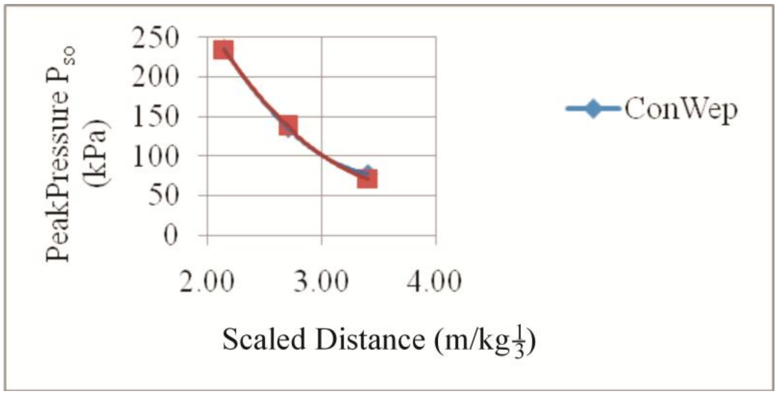

Figure 3: Comparison of ConWep Results with Experimental Output for Pso

Shock Wave Front Arrival Time

The best fitted curve yields the following equation.

$$T_a = \frac{4.637}{C_a}\left(\frac{R}{Q^{1/3}}\right)^{-0.138}$$

Where T_a is in sec, R is the range in meters and Q is the TNT charge weight in kg. C_a is the velocity of sound in air taken as 340m/s. Previous research by Adeel gives following equation.

$$T_a = \frac{6.46}{C_a}\left(\frac{R}{Q^{1/3}}\right)^{-0.41}$$

Table 6: Comparison of Empirical Equations for Ta

TNT Weight (kg)	Range (m)	Scaled Distance ($m/kg^{1/3}$)	Ta	
			Adeel Eq. (msec)	Current Research (msec)
0.6	2.87	3.4	11.5	11.52
1.2	2.87	2.7	12.6	11.89
2.4	2.87	2.14	13.9	12.27

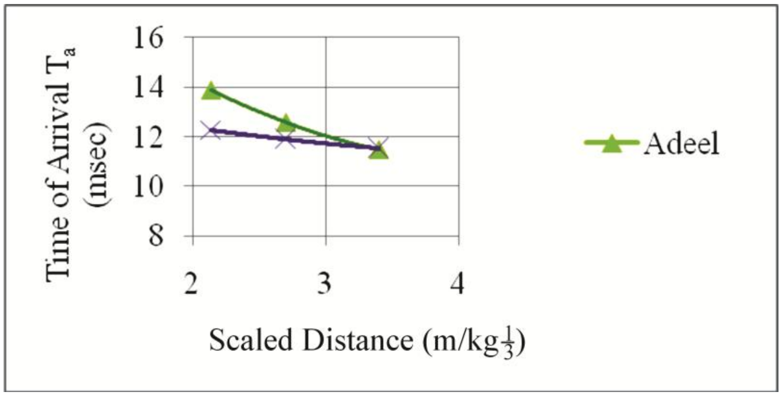

Figure 4: Graph Showing Comparison of Empirical Equations for Ta

Positive Phase Duration of the Shock Wave

The duration of the positive phase of the shock wave is the summation of the rising time to the maximum pressure and the decreasing time to the ambient pressure.
So

$$T_a = T_r + T_d$$

The experiments yield following relationships

$$T_r = 0.3108 \left(\frac{R}{Q^{1/3}}\right)^{-0.1}$$

$$T_d = 2.462 \left(\frac{R}{Q^{1/3}}\right)^{-0.472}$$

Therefore T can be written as

$$T = 0.3108 \left(\frac{R}{Q^{1/3}}\right)^{-0.1} + 2.462 \left(\frac{R}{Q^{1/3}}\right)^{-0.472}$$

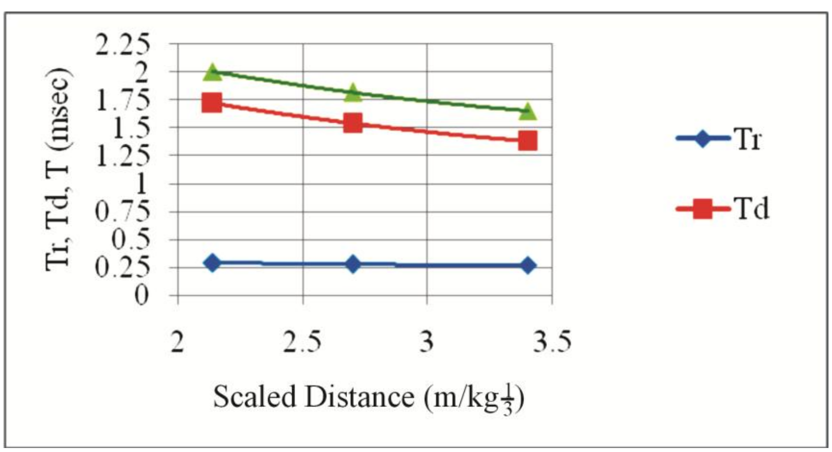

Figure 5: Graph Showing Positive Phase Duration of the Shock Wave

Peak Particle Acceleration

The experiments yield following equation for the PPA against scaled distance

$$PPA = 250.43 \left(\frac{R}{Q^{1/3}}\right)^{-2.361}$$

Where PPA is in terms of acceleration of gravity in m/s^2, R is in meters and Q is TNT equivalent charge weight in kg. The experimental results are compared with that obtained from ConWep.

Table 7: Comparison of ConWep and Experimental Output for PPA

Explosive Weight (kg)	Range (m)	Scaled Distance (m/kg$^{1/3}$)	PPA, g's (m/s^2)	
			ConWep	Experimental
0.6	2.87	3.40	13.68	13.5
1.2	2.87	2.70	23.81	25.4
2.4	2.87	2.14	41.45	40.2

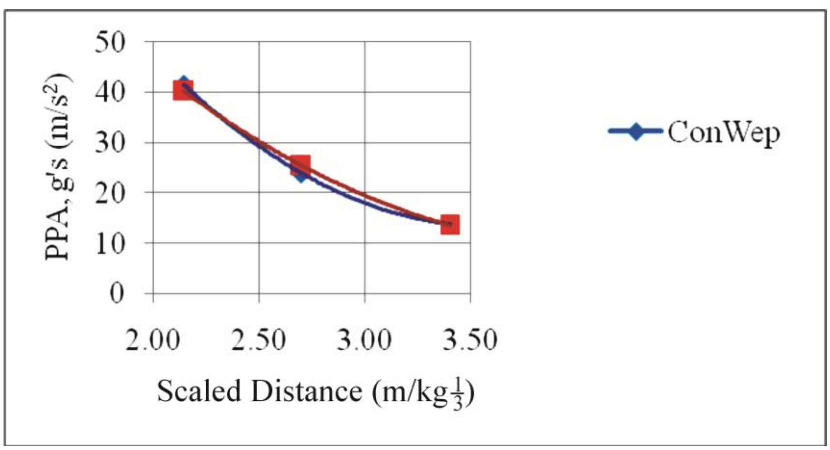

Figure 6: Comparison of ConWep Results with Experimental Output for PPA

Time of Arrival of the Ground Shock

The experimental values of time of arrival of the ground shock with respect to the scaled distance are shown in below table.

Table 8: Time of arrival of the Ground Shock ta

Explosive Weight (kg)	Range (m)	Scaled Distance ($m/kg^{1/3}$)	t_a (msec)
0.6	2.87	3.40	14.8
1.2	2.87	2.70	14.83
2.4	2.87	2.14	15.0

These values when plotted give the following empirical equation.

$$t_a = \frac{23.3}{C_s}\left(\frac{R}{Q^{1/3}}\right)^{-0.029}$$

Where t_a the ground shock arrival time in sec, R is the range in meters, Q s the TNT equivalent charge weight in kg and C_s is the soil seismic velocity which is taken as 1524m/s for the saturated sandy soil (soil considered for the current study).

Time Lag between Ground Shock and the Blast Wave Arrival Time

$$T_{lag} = T_a - t_a$$

$$T_{lag} = \frac{4.637}{C_a}\left(\frac{R}{Q^{1/3}}\right)^{-0.138} - \frac{23.3}{C_s}\left(\frac{R}{Q^{1/3}}\right)^{-0.029}$$

Structural Design of the RCC Wall

RCC wall is designed for blast loading using the methodology described in UFC_3_340_02.
Calculation is shown below,

1. Establish design parameters

a). Refer to fig 4-17[1], maximum support rotation equal to 2 degrees and cross-section type I.

b). L = 100 inches, H = 80 inches

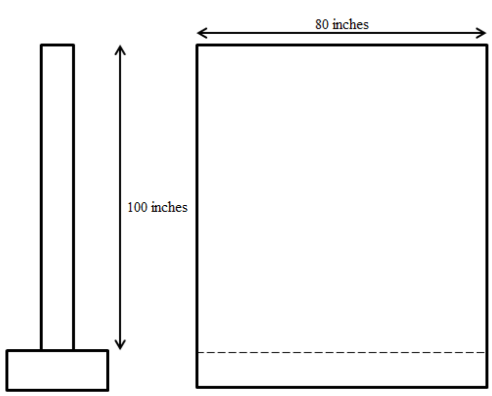

Figure 7: Wall Dimensions

c). Pressure-Time Loading

Charge Weight = 4 kg

TNT Equivalent, Q = 0.6 x 4 = 2.4 kg = 5.3 lb

Apply 20% safety factor, so Q = 6.36 lb

Range, R = 2.87 m = 9.416 ft

Scaled Distance, Z = $\frac{R}{\sqrt[3]{Q}}$

=> Z = 5.08 $ft/lb^{1/3}$

Using fig 2-15[1], following values are interpreted

P_{so} = 40 psi

$\frac{t_o}{Q^{1/3}}$ = 1.65 $ms/lb^{1/3}$

=> t_o = 3.05 ms

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1. *UFC_3_340_02*

So, the pressure-time loading diagram will be as follows

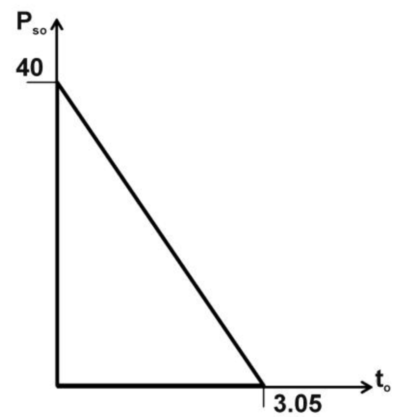

Figure 8: Blast Load

2. Select a cross-section of element including thickness and concrete cover over the reinforcement. Determine the static design stresses for concrete and the steel.

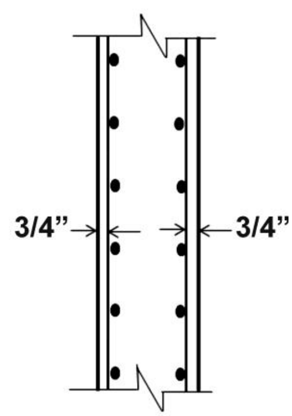

Figure 9: Reinforced Concrete Wall Cross-section

Assume wall thickness T_c = 6.5 inches

Concrete Cover = ¾ inches

Static Stress of Concrete f'_c = 4000 psi
Static Stress of Reinforcement f_y = 50000 psi
3. Determine the dynamic increase factor for the concrete and steel using table 4-1[1]
Concrete:
Diagonal Tension = 1.00
Reinforcement:
Bending = 1.17

1. *UFC_3_340_02*

Diagonal Tension = 1.0
Direct shear = 1.10
4. Determine the dynamic strength of materials
Concrete (f'_{dc}):
Diagonal Tension 1.00 x 4000 = 4000 psi
Reinforcement (f_{dy}):
Bending 1.17 x 50000 = 58500 psi
Diagonal Tension 1.0 x 50000 = 50000 psi
Direct Shear 1.10 x 50000 = 55000 psi
5. Determine the dynamic design stresses using table 4-2[1]
Concrete (f'_{dc}):
Diagonal Tension = 4000 psi
Reinforcement ($f_{ds} = f_{dy}$ for $\theta < 2$):
Bending = 58500 psi
Diagonal Tension = 50000 psi
Direct Shear = 55000 psi
6. Assume Reinforcement in vertical direction = No.4 @ 8 in c/c
Assume Reinforcement in horizontal direction = No.4 @ 12 in c/c
7. Calculate d and the steel ratios for each direction.

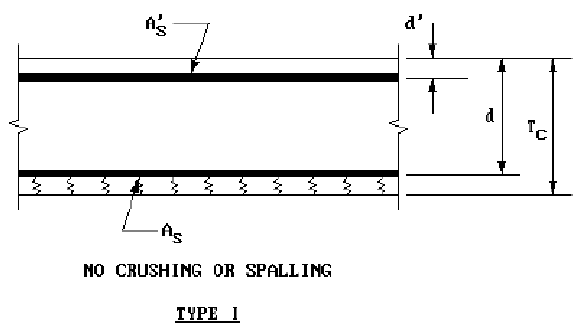

Figure 10: Type I Cross-Section

Let's assume No.3 stirrups.

d_V = 6.5 – 0.75 – 0.5/2 – 0.375
= 5.125 in

1. *UFC_3_340_02*

d_H = 6.5 – 0.75 - 0.5 – 0.375 – 0.5/2
= 4.625 in

$\rho_V = \frac{A_{sv}}{bd_v}$

$= \frac{0.29}{12 \times 5.125}$

$= 0.0047$

ρ_H $= \frac{A_{sH}}{bd_H}$

$= \frac{0.20}{12 \times 4.625}$

$= 0.0036$

Use table 4-3[1] to calculate the minimum reinforcement ratios.

ρ_{min} in vertical direction $= \frac{1.875\sqrt{f_c}}{f_y}$

$= \frac{1.875\sqrt{4000}}{50000}$

$= 0.0024$

Which is less than ρ_V so O.K.

ρ_{min} in horizontal direction $= \frac{1.25\sqrt{f_c}}{f_y}$

$= \frac{1.25\sqrt{4000}}{50000}$

$= 0.0016$

Which is less than ρ_H so O.K.

8. Calculate the moment capacities of both the positive and negative reinforcement in both directions.

Use equation $M_u = \frac{A_s f_{ds}}{b}\left[d - \frac{a}{2}\right]$

In which $a = \frac{A_s f_{ds}}{0.85\, bf_{dc}}$

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1. *UFC_3_340_02*

Now, moment in vertical direction

$$M_{VN} = M_{VP} = \frac{0.29x58500}{12}\left(5.125 - \frac{0.29x58500}{0.85x12x4000}\right)$$

$= 6660$ in-lbs/in

And moment in horizontal direction

$$M_{HN} = M_{HP} = \frac{0.20x58500}{12}\left(4.625 - \frac{0.20x58500}{0.85x12x4000}\right)$$

$= 4230$ in-lbs/in

9. Establish the values of r_u.

Using table 3-1[1]

$r_u = \frac{2M_N}{L^2}$

$= \frac{2x6660}{(100)^2}$

$= 1.33$ psi

10. Determine the modulus of elasticity of concrete and steel

a). Modulus of elasticity of materials.

Modulus of elasticity of concrete $E_c = W_c^{1.5}\, 33(fc)^{1/2}$

$= (150)^{1.5}\, 33\, (4000)^{1/2}$

$= 3834250$ psi

Modulus of elasticity of steel $E_s = 29 \times 10^6$ psi

b). Modular Ratio.

Modular Ratio n $= \frac{E_s}{E_c}$

$= 7.5$

11. Determine the average moment of inertia for an inch strip.

a). Gross moment of inertia I_g $= \frac{T_c^3}{12}$

$= \frac{(6.5)^3}{12}$

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1. *UFC_3_340_02*

$= 23\ in^4/in$

b). Moment of inertia of cracked section

Avg. depth for vertical direction, $d_{v(avg)}$ $= 5.125$ in

Avg. depth for horizontal direction, $d_{H(avg)}$ $= 4.625$ in

Avg. ρ in vertical direction, $\rho_{V(avg)}$ $= \frac{A_s}{bd_{V(avg)}}$

$= 0.0047$

Avg. ρ in horizontal direction, $\rho_{H(avg)}$ $= \frac{A_s}{bd_{H(avg)}}$

$= 0.0036$

Use fig 4-11[1], to find the coefficient of inertia of cracked section

Hence F for vertical direction = 0.0246

And F for horizontal direction = 0.0193

So, I_{cV} $= Fd_{V(avg)}^3$

$= 0.0246 \times 5.125^3$

$= 3.31\ in^4/in$

And I_{cH} $= Fd_{H(avg)}^3$

$= 0.0193 \times 4.625^3$

$= 1.91\ in^4/in$

c). Average moment of inertia of cracked section

I_c $= \frac{LI_{cV} + HI_{cH}}{L + H}$

$= \frac{(100x3.31) + (80x1.91)}{100 + 80}$

$= 2.69\ in^4/in$

d). Average moment of inertia I_a $= \frac{I_c + I_g}{2}$

$= 12.845\ in^4/in$

12. Establish the equivalent elastic stiffness

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1. *UFC_3_340_02*

Using table 3-8[1], K_E $= \frac{8EI}{L^4}$

$= \frac{8x3834250x12.845}{(100)^4}$

$= 3.94$ psi/in

13. Establish the value of elastic deflection

XE $= \frac{r_u}{K_E}$

X_E $= \frac{1.33}{3.94}$

X_E = 0.338 in

14. Calculate the effective mass of element

a). Load mass factors

use table 3-12[1], Elastic K_{LM} = 0.65

Plastic K_{LM} = 0.66

Average load mass factor K_{LM} = 0.655

b). unit mass of an element m $= \frac{wT_c}{g}$

$= \frac{150x6.5}{32.2x12^3x12}$

= 0.00146 psi-sec^2/in

= 1460 psi-ms^2/in

c). Effective unit mass m_e = K_{LM} x m

= 0.655 x 1460

= 956.3 psi-ms^2/in

15. Calculate the natural period of vibration.

$$TN = 2\pi\sqrt{\frac{m_e}{K_E}}$$

$$= 2\pi\sqrt{\frac{956.3}{3.94}}$$

1. *UFC_3_340_02*

= 97.9 ms

16. Determine the maximum response of element

a). Response Chart parameters:

Peak Pressure, P = 40 psi

Peak resistance, r_u = 1.33 psi

Duration of Blat load, T = 3.05 ms

Period of vibration T_N = 97.9 ms

P/r_u = 30

T/T_N = 0.03

b). using fig 3-64(a)[1]

X_m/X_E= 4.2

X_m = 4.2 x 0.338

= 1.42 in

17. Check the support rotation assumed.

Using table 3-5[1] X_m = Ltanθ

=> 1.42 = 100 x tanθ

=> tanθ = 0.0142

=> Θ = 0.81°

Which is less than 2° as assumed in step no.1. so assumed section is O.K.

18. Check diagonal tension at supports.

a). Calculate ultimate shear stress at support by dividing the values of the support shear from table 3-9[1].

So, $V_{uV} = \frac{r_u L}{d_v}$

$$= \frac{1.33 \times 100}{5.125}$$

$$= 25.95 \text{ psi}$$

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1. *UFC_3_340_02*

$$V_{uH} = \frac{r_u L}{d_H}$$

$$= \frac{1.33 \times 100}{4.625}$$

$$= 28.75 \text{ psi}$$

b). Allowable shear stress

since $V_c = \left[1.9\sqrt{f_{dc}} + 2500\rho\right] \leq 3.5\sqrt{f_{dc}}$

so $V_{cV} = 1.9\sqrt{4000} + 2500 \times 0.0047$

$$= 132 \text{ psi} \leq 3.5\sqrt{4000}$$

$$V_{cH} = 1.9\sqrt{4000} + 2500 \times 0.0036$$

$$= 129 \text{ psi} \leq 3.5\sqrt{4000}$$

Since $V_{uV} < V_{cV}$

And $V_{uH} < V_{cH}$

So, the design of shear reinforcement will be based on the minimum requirement of the shear reinforcement.

Referring to table 4-4[1], for the case under consideration no stirrups are required.

Conclusions

1. If the horizontal range is kept constant, it is found experimentally that the numerical equation gives similar results as that provided by Brode and Henrych for small charge weights.
2. ConWep output and experimental values of peak pressure are in close agreement with each other.
3. For same horizontal range, larger the charge weight lesser will be the time of arrival of the shock wave.
4. For saturated sandy soil, the values of PPA obtained experimentally and analytically with ConWep are in close relation with each other.
5. The time lag between the ground shock and the blast wave shows that the air blast wave

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1. *UFC_3_340_02*

reaches the structure before the arrival of ground shocks for small scaled distances say less than 5m.

6. The best economical solution for the blast resistant R.C.C structures is to maximize the stand-off distance for important buildings. As far as the structural design is concerned, the best approach is to prevent the progressive collapse.

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