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Stress Distribution of Stiffened Slabs Subjected to Localized Friedlander Load

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ABSTRACT: **Abstract:*****Localized blast loadings are often overlooked during the structural system design process despite their potential detrimental effects. In this research, a set of slab models with semi-rigid boundary conditions on its sides and with variations in geometry and stiffness are subjected to a Friedlander localized blast loading at certain locations. The main system responses that are observed are the transversal deflections at midspan and the internal stresses of the system, particularly the maximum principle stress, minimum principle stress, and the maximum shear stress. Three loading phases are included in the analysis, namely: the positive phase, the negative phase, and the free vibration phase. Analyses are carried out utilizing a numeric approach termed the Modified Bolotin Method. Deflections resulting from various load positions on the set of slab models throughout all three phases are then compared. Stresses are calculated on all slab models with the Friedlander localized blast loading applied at midspan and the results are presented as stress contours that are then compared between each model. Based on the results from this research, adding 2cm to the slab thickness provides a better structural response compared to adding 2 secondary beams to one of the orthogonal directions of the slab.***

Keywords: Friedlander blast load, stiffener, Modified Bolotin Method, principle stress

1.Introduction

Floor slabs are one of the most vital structural elements in construction. Almost every project contains slabs with various conditions and environments and they can be found in hotels, schools, apartments, malls, shelters, roads, and other public facilities. Due to this, slabs should be evaluated at different conditions and situations so that we can achieve a safe, economic, and functional design. Some of the slabs responses analyzed include deflections, internal forces, and stresses so that the design will satisfy both strength and serviceability requirements [1].

The types of loads commonly modeled in slab analysis are gravity loads (dead and live loads) and lateral loads (wind and seismic loads), however, many engineers ignore the effects from other dynamic loads such as machine vibrations or blast loads. An effective slab design should anticipate any load that potentially may affect the slab, including blast loads,which have unique characteristics and effects on slabs that must be taken into account. An explosion can be defined as the rapid release of energy as mass from a reactive material that is then converted into an extremely dense region of high pressure gas[2]. Friedlander load is a semi-empircal localized blast load that is modeled using an exponential function for the positive phase and a different function for the negative phase. The negative phase of a blast load is often ignored in analysis; however recent studies show that the negative phase of the localized dynamic load plays an important factor in increasing the maximum structure responses [3]. The dynamic behavior of a stiffened damped orthrotropic plate has already been a subject of interest for several years [4, 5] and those studies are done with a few variations on the object such as total number of stiffeners used and the characteristic material of the plate element utilized.

Previous research included a ground floor slab with semi-rigid supports and an elastic Pasternak foundation [6]. The slab is loaded with a Friedlander load that follows the parameters used by Susleret. al[7]. Furthermore, that research compared the differences of the responses of a structural slab supported on soft soil, medium soil, and hard soil.

Any element given a dynamic excitation will show unique characteristics and behaviors based on its relative stiffness and mass. The dynamic behavior can be quantitized by finding the natural frequency (eigenvalue) and the mode shape (eigenvector) of the structure. The Modified Bolotin Method is a method that can be used to obtain the vibrational natural modes. This method has the advantage of being able to solve the differential equation of plates accurately for higher modes [8].

In this research, an elastic and orthrotropic floor slab under a localized blast load (Friendlander) is analyzed. The governing differential eqution for an orthogonal damped thin rectangular plate that is given a dynamic transversal load $p_z(x,y,t)$ is derived using the Newton's Second Law which states that all forces acting on a body must be under equilibrium. Solving the equation is achieved by finding a unique solution for the equation of motion used for an undamped structure ($\gamma = 0$) and calculating the eigenvalues or natural frequencies of the structure. First, the geometric boundary conditions that apply to the model must be determined. Choosing how the slab supports are modeled is very critical because of the large impact it has on the desired responses. Slabs that are simply supported have often been a subject under research because of the relative ease of analysis [8], however this support condition does not accurately represent the conditions found in-situ. It turns out that these slabs simultaneously experience bending moments and rotations at the supports, therefore, floor slabs should be modeled with semi-rigid supports, not simply supported or fully rigid.

2.Literature Survey

This research analyzes a few models of slabs with and without floor beams and also variations in slab thickness. Because floor beams are present, the slab is orthrotropic, meaning that the stiffness values in both orthogonal directions are different. The floor slab analyzed is elastic linear and damped, therefore the governing differential equation is given by the following equation [9].

$$D_x \frac{\partial^4 w}{\partial x^4} + 2B \frac{\partial^4 w}{\partial x^2 \partial y^2} + D_y \frac{\partial^4 w}{\partial y^4} + \rho h \frac{\partial^2 w}{\partial t^2} + \xi h \frac{\partial w}{\partial t} = p_z(x,y) \quad (1)$$

For rectangular orthotropic slabs stiffened by floor beams, the equation above can be used to obtain the plate responses from certain loads. The geometric parameters that should be determined are shown in Fig. 1.

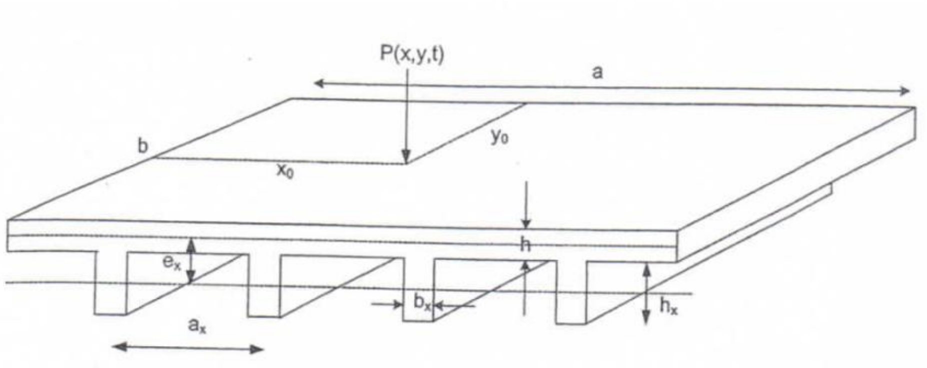

Figure 1: Rectangular orthotropic slabs with floor beams[8]

Generally, the analysis is carried out under static loads where the acceleration and velocity of the loads are insignificant. Structures that are given dynamic loading with intensities that are a function of time will yield responses differentfrom static loading. In order to obtain the values of the structural reponses under dynamic loading, the solution to the following differential equation of motion must be completed.

$$m\ddot{u} + c\dot{u} + ku = p(t) \quad (2)$$

A structure under dynamic loading will have two responses depending on the form of excitation; those responses are free vibration and forced vibration. Free vibration is the response of the structure after given a certain initial condition, such as a displacement in a specified direction, and the structure will vibrate due to those initial conditions. The response from the free vibration component can be obtained by solving for the homogenous solution for the above differential equation. Forced vibration is the dynamic response from any direct loading on the structure. The response from the forced vibration component can be obtained by solving for the particular solution for the differential equation of motion. The total reponse is the sum of the reponses from the free vibration and forced vibration components.

Blast loads have an arbitruary function, which requires the load function to be discretized and solved for using Duhamel's Integral in order to obtain the total response for a certain duration *t*[10].

$$u(t) = \frac{1}{m\,\omega_D} \int_0^t p(\tau) e^{-\xi \omega_n (t-\tau)} \sin[\omega_D (t-\tau)] \quad (3)$$

A few studies have developed the equation for a localized Friedlander load for both the positive phase and negative phase of the load. The positive phase of a blast load is commonly expressed as a linear or exponential function and acts in the direction towards the slab. In this case, the localized Friedlander load utilizes an exponential function which is as follows.

$$p_r(t) = p_{r,max} \left(1 - \frac{t}{t_d}\right) e^{-bt/t_d} \quad (4)$$

After the positive phase is completed, the load enters its negative phase, changing into a vacuum force. Generally this phase has amplitude smaller than the positive phase but it has a much longer duration, approximately twice the length of the positive phase. The function of a Friedlander load for both the positive and negative phase is shown in Fig. 2.

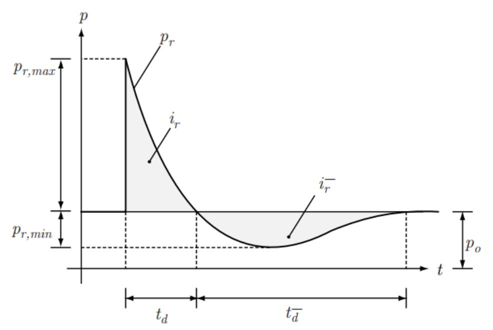

Figure 2: Function of a Friedlander load [2]

3.Research Methodology

The slab models in this study are rectangular and are assumed to have a uniform thickness and also floor beams in one of the orthogonal directions for two out of four models. Based on the study by Alisjahbana and Wangsadinata [4,5], the differential equation to find the responses for the slab is given as below:

$$D_x\left(\frac{\partial^4 w(x,y,t)}{\partial x^4}\right)+2B\left(\frac{\partial^4 w(x,y,t)}{\partial x^2\partial y^2}\right)+\gamma h\left(\frac{\partial w(x,y,t)}{\partial t}\right)+D_y\left(\frac{\partial^4 w(x,y,t)}{\partial y^4}\right)+\rho h\left(\frac{\partial^2 w(x,y,t)}{\partial t^2}\right)=p_z(x,y,t) \qquad (5)$$

Where D_x is the flexural stiffness in the direction perpendicular to the floor beam, D_y is the flexural stiffness in the direction parallel to the floor beam, B is the torsional stiffness, γ is the damping coefficient, ρ is the density of the slab, and p_z is the dynamic transversal load. The values of D_x, D_y, and B can be calculated using the following equations.

$$D_x=\frac{E'h^3}{12}+\frac{E_x b_x}{6a_x}\left(\left(h_x-\left(e_x-\frac{h}{2}\right)\right)^2(2h_x+e_x+h)-\left(e_x-\frac{h}{2}\right)^2(e_x+h)\right) \qquad (6)$$

$$D_y=\frac{E'h^3}{12} \qquad (7)$$

$$B=\sqrt{D_x D_y} \qquad (8)$$

Next, the boundary conditions determined according to the mathematical model are used.In the model, the four edges are given semi-rigid supports where there are no vertical nor horizontal displacements, however there is a rotation that occurs simultaneously with a certain bending moment. Based on this condition, the boundary conditions can be formulated as given below.

$$w(x,y)=0 \quad di\ x=0, x=a, y=0, y=b \qquad (9)$$

$$m(x,y)=-D_x\left[\frac{\partial^2 w(x,y)}{\partial x^2}+v_y\frac{\partial^2 w(x,y)}{\partial y^2}\right]=k_1\frac{\partial w(x,y)}{\partial x} \quad di\ x=0, x=a \qquad (10)$$

$$m(x,y)=-D_y\left[\frac{\partial^2 w(x,y)}{\partial y^2}+v_x\frac{\partial^2 w(x,y)}{\partial x^2}\right]=k_2\frac{\partial w(x,y)}{\partial y} \quad di\ y=0, y=b \qquad (11)$$

where k_1 and k_2 are the spring rotational constants in the x and y direction respectively. Based on the boundary conditions applied, a trial function is selected for the governing differential equation.

$$W_{mn}=A_{mn}\sin\left(\frac{m\pi x}{a}\right)\sin\left(\frac{n\pi x}{b}\right) \qquad (12)$$

The trial function is substituted into equation (5) and is then algebraically manipulated to obtain equation (13).

$$\omega_{mn}^2=\frac{\pi^4}{\rho h}\left[D_x\left(\frac{m}{a}\right)^4+2B\left(\frac{mn}{ab}\right)^2+D_y\left(\frac{n}{b}\right)^4\right] \qquad (13)$$

Equation (13) above gives the eigenvalue of the structure for a certain mode shape. By taking the positive square root of the eigenvalue, the natural frequency of the structure is obtained. Equation (13) is only valid if the slab is simply supported on all sides, whereas the object of this study applies semi-rigid boundary conditions, which requires further modification of the above equations.

The Modified Bolotin Method is a numeric approach to solving the differential equations of a plate with semi-rigid boundary conditions while taking into account the effects of higher modes. The eigenvalue for a rectangular slab with semi-rigid supports on all sides can be found by solving the equation in a manner similar to a simply supported plate. The integer index values for the x and y directions are substituted with coefficients *p* and *q* for the mode shapes in the x and y directions respectively. The values of p and q are real numbers that are obtained by solving the auxiliary problems. The eigenvalue equation for a slab with semi-rigid supports is expressed by equation (14).

$$\omega_{mn}^2 = \frac{\pi^4}{\rho h}\left[D_x\left(\frac{p}{a}\right)^4 + 2B\left(\frac{pq}{ab}\right)^2 + D_y\left(\frac{q}{b}\right)^4\right] \tag{14}$$

The values for p and q are obtained by the Modified Bolotin Method through solving two transcendental equations, one for each orthogonal direction. Both transcendental equations are obtained by the determinant of two matrices of the imposed boundary conditions for both the x and y directions. Because the auxiliary problems are solved for each direction seperately, ignoring any effects of the perpendicular direction, this problem can be categorized as a Levy type problem.

For the x and y directions, each have a Levy type problem called auxiliary 1 and auxiliary 2. Auxiliary 1 will provide the solution for the position function X(x) in the x direction while the y direction will only vibrate harmonically. On the other hand, auxiliary 2 will provide the solution for the position function Y(y) in the y direction while the x direction will only vibrate harmonically.

The solution for first Levy auxiliary is solved for the x direction by utilizing the following trial function

$$W(x,y) = X(x)\sin\left(\frac{q\pi y}{b}\right) \tag{15}$$

where X(x) is the position function from the orthrotropic plate in the x direction. The characteristic equation above can be solved by assuming the following:

$$X(x) = Ae^{\lambda x} \tag{16}$$

The above trial function is used to solve for the characteristic equation which yields two real roots and two imaginary roots. The solution for the first auxiliary problem is expressed as:

$$\begin{aligned} X(x) = {} & A_1\cosh\left(\frac{\beta\pi x}{ab}\right) + A_2\sinh\left(\frac{\beta\pi x}{ab}\right) \\ & + A_3\cos\left(\frac{p\pi x}{a}\right) + A_4\sin\left(\frac{p\pi x}{a}\right) \end{aligned} \tag{17}$$

where A_1, A_2, A_3, and A_4 are coefficients determined by imposing the boundary conditions at $x=0$ and $x=a$. The solutions from the above equations must be non-trivial in order to find the eigenvalues for the structure under loading. The non-trivial values of p and q are acquired by setting the determinant of the characteristic equations equal to 0 as shown in equation (18).

$$\begin{vmatrix} 1 & 0 & 1 & 0 \\ F_1 & \frac{\beta}{ab}k_1 & -F_2 & \frac{p}{a}k_1 \\ C_1 & S_1 & c_1 & s_1 \\ F_1C_1 + \frac{\beta}{ab}S_1k_1 & F_1S_1 + \frac{\beta}{ab}C_1k_1 & -(F_2c_1 + \frac{p}{a}s_1k_1) & -(F_2s_1 - \frac{p}{a}c_1k_1) \end{vmatrix} = 0 \tag{18}$$

Equation (18) above is used to solve for the real roots p and q that will be used to obtain the eigenvalues of the structure. After the values for p and q are obtained, the coefficients A_1, A_2, A_3, and A_4 can be calculated as well. The values of A_2, A_3, and A_4 are normalized with respect to A_1 and by substituting the values of A_1, A_2, A_3, and A_4, the position function for the first auxiliary in the x direction is shown in equation (19).

$$\begin{aligned} X(x) = {} & \cosh\left(\frac{\pi\beta x}{ab}\right) + \\ & \frac{b(c_1k_1p - C_1k_1p + a(F_1 + F_2)s_1)}{k_1(bpS_1 - s_1\beta)}\sinh\left(\frac{\pi\beta x}{ab}\right) \\ & - \cos\left(\frac{\pi px}{a}\right) \\ & - \frac{ab(F_1 + F_2)S_1 + (c_1 - C_1)k_1\beta}{k_1(bpS_1 - s_1\beta)}\sin\left(\frac{\pi px}{a}\right) \end{aligned} \tag{19}$$

The second auxiliary in the y direction can be solved by a procedure analogous to the above derivations.

As stated previously, the slab responses for a dynamic transversal load requires two distinct solutions, the homogenous solution (free vibration) and the particular solution (forced vibration). The homogeneous and particular solutions are solved separately and both solutions are added to obtain the total structure response. The homogeneous solution, w_H, is the solution for the structure that is excited by an initial deformation or velocity (initial condition) that causes the structure to respond. In order to solve for the homogenous solution, the right side of the governing equation of motion is set equal to zero. The separation of variables method is used to solve for governing differential equation of motion. This method will simplify solving for the above equations by separating the governing differential equation into two different equations: the spatial differential equation *W(x,y)* and the temporal differential equation *T(t)*. The spatial differential equation is a function of position of the load at *x* and *y* while the temporal differential equation is a function of time *t*. The homogeneous solution can be expressed as the following equation.

$$w_H = w(x,y,t) = \sum_{p=1}^{\infty}\sum_{q=1}^{\infty} W_{pq}(x,y)T_{pq}(t)$$
$$= \sum_{p=1}^{\infty}\sum_{q=1}^{\infty}[X_{pq}(x)Y_{pq}(y)]e^{-\xi\omega_{pq}t}[a_0\cos(\omega_D t) + b_0\sin(\omega_D t)] \quad (20)$$

Similary to the method for solving the homogeneous solution, solving for particular solution also requires the separation of variables method. The coefficients in the above equation are adjusted to take into account the effects from load $p_z(x,y,t) \neq 0$ that was ignored in the homogeneous solution. The homogenous solution $\hat{T}_{pq}(t)$ contains constants that are calculated based on the initial conditions because this function uses a transient vibration without loading. Meanwhile, the particular solution $T^{*}_{pq}(t)$ depicts the vibrations due to an acting load. The particular solution from the differential equation is the total of the spatial and temporal components and is expressed as follows.

$$w_p = \sum_{p=1}^{\infty}\sum_{q=1}^{\infty} X_{pq}(x)Y_{pq}(y) \quad (21)$$

The total solution is the actual occurring response that takes into account the effects from both free and forced vibrations. This solution is obtained through summing the homogeneous and particular solutions, thereby acquiring the following equation.

$$\mathrm{w} = w_H + w_P \quad (22)$$

The blast load used in this study is a localized blast load developed by Friendlander, with the positive phase using an exponential function and the negative phase using the cubic negative phase developed by Ganström [11]. Based on the study of Rigby et. al [2], for the negative phase of a Friedlander load, the cubic negative phase approach yields the most accurate results. The parameters used for the Friedlander load are identical to the ones used in a study by Susleret. al[7].Those parameters are tabulated in Table 1.

Table 1: Friedlander load parameters

Notation	Value	Unit	Description
P_{max}	28906	Newton [N]	Positive phase amplitude
P_{min}	7226,5	Newton [N]	Negative phase amplitude
t_A	0	Second [s]	Initial load time
t_P	0,0018	Second [s]	Positive phase duration
t_N	0,0036	Second [s]	Negative phase duration
α	0,35	-	Wave shape coefficient

Influences due to the location of the Friedlander load at any arbitrary position that changes with time are expressed with Dirac's Delta function.

$$P(x,y,t) = P[x(t),y(t),t]$$
$$= P(t)\delta[x - x(t)]\delta[y - y(t)] \quad (23)$$

4.Results and Discussion

Four plate models in this study are loaded using a localized blast load that changes in intensity over time, thereby resulting in structure responses that vary greatly over time as well. Resulting slab deflections in midspan are plotted with respect to time in order to produce the time history of slab deflections within a predetermined interval. The duration displayed is from 0 seconds to 0.25 seconds so that all the deflections from all three phases are included.

The absolute maximum deflection is the largest deflection that occurs throughout the duration of the loading and after the loading is no longer applied (free vibration). Absolute maxmimum deflections are critical in design and are required to check to see if the serviceability and small deflection requirements are satisfied.Figure 3 below displays the response history of all slab models with the load at three different positions. Tables 2 to 4 show the values of the maximum and minimum deflections for each plate model with various load positions.

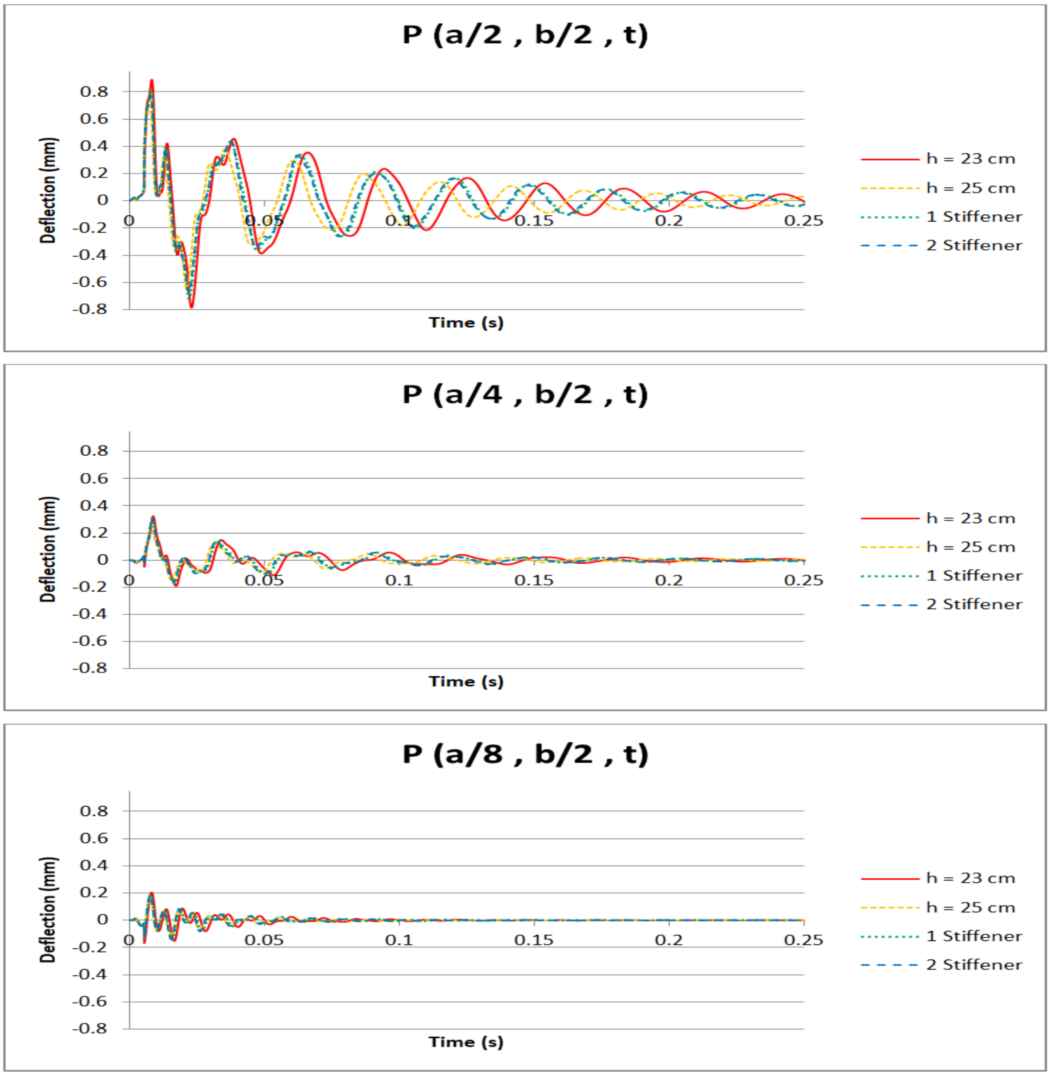

Figure 3: Dynamic deflections at midspan for three load positions

Table 2: Absolute deflections with load at midspan

Absolute Deflection, Load atMidspan (mm)								
Model	**h = 23cm**		**h = 25cm**		**h = 23cm, 1 Stiffener**		**h = 23cm, 2 Stiffener**	
	Maximum	**Minimum**	**Maximum**	**Minimum**	**Maximum**	**Minimum**	**Maximum**	**Minimum**
Positive Phase	0.02394	6.65E-49	0.02037	9.445E-49	0.02341	-3.37E-17	0.02279	-3.37E-17
Negative Phase	0.08079	0.002899	0.09089	0.00424	0.09176	0.00422	0.09403	0.00356
Free Vibration	0.88998	-0.7853	0.7418	-0.6506	0.81783	-0.7198	0.77365	-0.67265

Table 3: Absolute deflections with load at one-forth of span

Absolute Deflection, Load at One-Forth of Span (mm)								
Model	**h = 23cm**		**h = 25cm**		**h = 23cm, 1 Stiffener**		**h = 23cm, 2 Stiffener**	
	Maximum	**Minimum**	**Maximum**	**Minimum**	**Maximum**	**Minimum**	**Maximum**	**Minimum**
Positive Phase	8.026E-47	-0.002691	8.866E-47	-0.002299	-2.04E-17	-0.002585	-2.03E-17	-0.002585
Negative Phase	0.02668	-0.02026	0.0332737	-0.016133	0.0319801	-0.019599	0.0329224	-0.019386
Free Vibration	0.3238391	-0.189974	0.2673293	-0.16003	0.3106137	-0.172333	0.3085424	-0.158669

Table 4: Absolute deflections with load at one-eighth of span

Absolute Deflection, Load at One-Eighth of Span (mm)								
Model	**h = 23cm**		**h = 25cm**		**h = 23cm, 1 Stiffener**		**h = 23cm, 2 Stiffener**	
	Maximum	**Minimum**	**Maximum**	**Minimum**	**Maximum**	**Minimum**	**Maximum**	**Minimum**
Positive Phase	0.0021394	4.741E-47	0.0015257	5.222E-47	0.0016709	-7.06E-18	0.0017236	-7.57E-18
Negative Phase	0.0127699	-0.042471	0.0110687	-0.034441	0.0136132	-0.040803	0.0122583	-0.039456
Free Vibration	0.2042123	-0.167571	0.1675167	-0.124228	0.1898689	-0.142698	0.1815482	-0.137945

From the above tables, it is shown that the closer the load is to midspan, the larger the absolute maximum deflection. Furthermore, for all models and all load locations, the absolute maximum deflection takes place during the free vibration phase. The absolute maximum deflection is 0.89 mm which occurs in the model of the 23 cm thick plate without any floor beams and loading at midspan. For all load locations, the largest reduction of slab deflections is achieved by adding 2 cm to the slab thickness. These results reveal that adding floor beams are not as effective as adding to slab thickness in reduced slab deflections.

After the characteristic equation of motion *w(x,y,t)* is obtained, the values for the stresses that occur in the plate can be calculated. Stress is a complex value that contains many components. For this research, the stresses calculated are the maximum principle stresses, minimum principle stresses, and maximum shear stresses based on the formulations by Mohr's Theory. In order to find those maximum stresses, we first need to calculate the stress components σ_x, σ_y, and τ_{xy}. The values for σ_x, σ_y, danτ_{xy} are obtained using equations that show the relationship between displacements and stress components [12]. For all stress calculations, the load is only modeled at the midspan of the plate.The resulting stress contours of the maximum and minimum principle stresses and maximum shear stresses are illustrated in Figures 4 to 6 below.

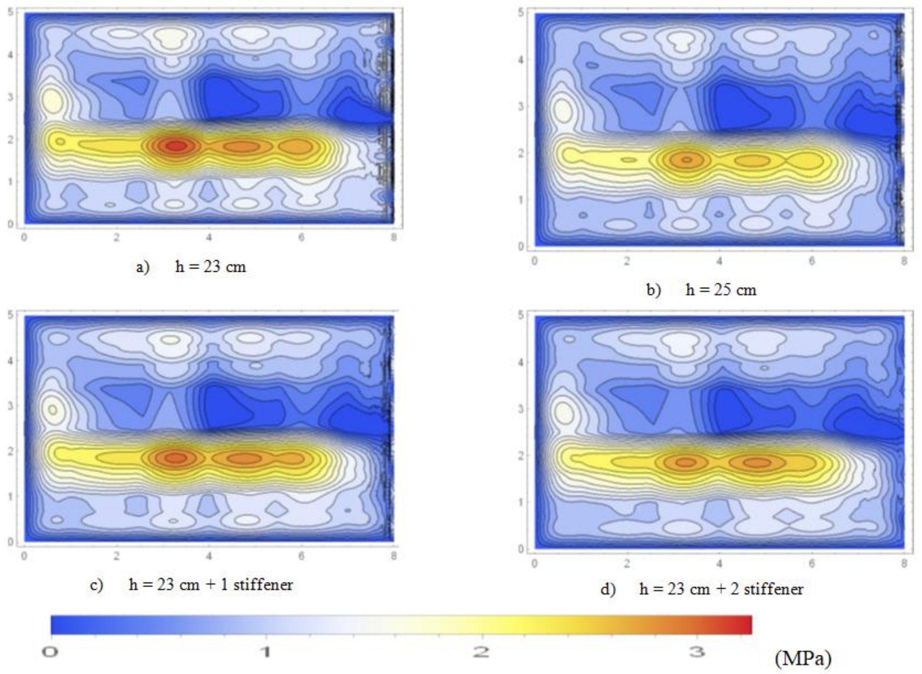

Figure4: Maximum principle stress contours

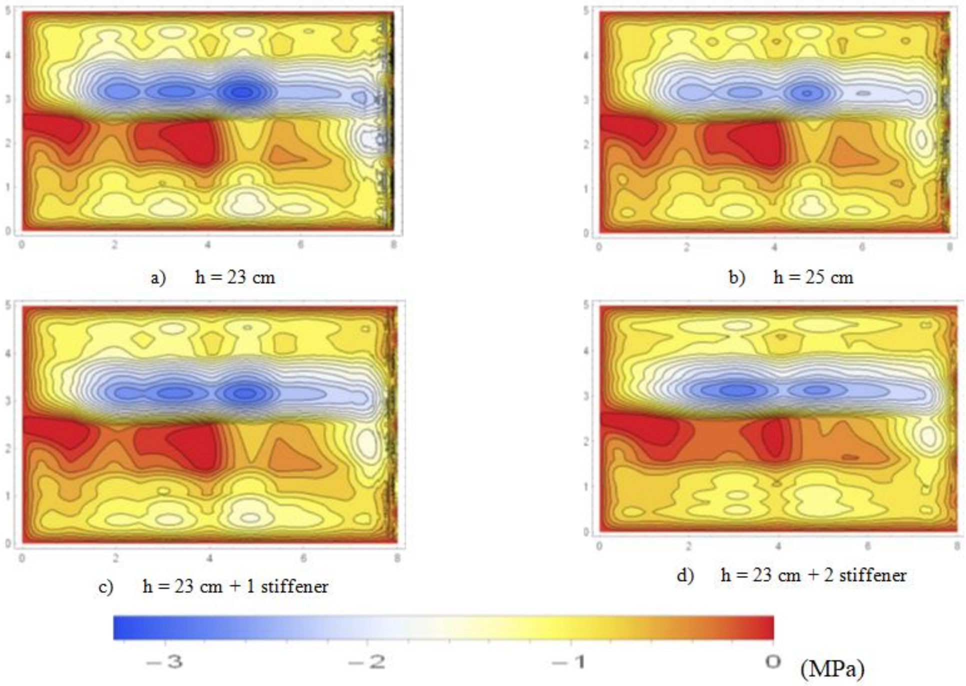

Figure 5: Minimum principle stress contours

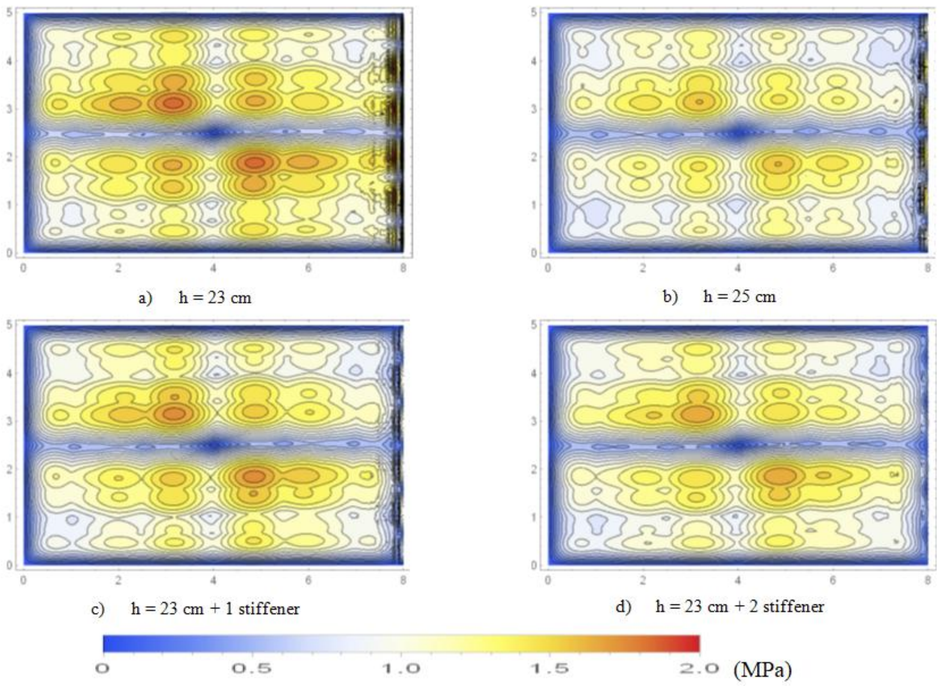

Figure 6: Absolute maximum shear stress contours

The values, coordinates, and time of the maximum principle stress, minimum principle stress, and maximum shear stress are tabulated in Tables 5-7 below.

Table 5: Maximum principle stress with load at midspan

Maximum Principle Stress (MPa)				
Model	**h = 23cm**	**h = 25cm**	**1 Stiffener (h=23cm)**	**2 Stiffener (h=23cm)**
x (m)	3.25	3.25	3.25	3.25
y (m)	1.8	1.8	1.8	1.8
t (s)	0.0084	0.0078	0.0082	0.0082
θ_p (°)	-13.6	-13.24	-9.667	-7.896
σ_{max}	3.14	2.7683	2.99554	2.829

Table 6: Minimum principle stress with load at midspan

Minimum Principle Stress (MPa)				
Model	h = 23cm	h = 25cm	1 Stiffener (h=23cm)	2 Stiffener (h=23cm)
x (m)	4.75	4.75	4.75	3.25
y (m)	3.2	3.2	3.2	3.2
t (s)	0.0084	0.0078	0.0082	0.0084
θ_p (°)	-3.148	-2.771	-2.99734	-2.82693

Table 7: Maximum shear stress with load at midspan

Maximum Shear Stress (MPa)				
Model	t = 23cm	t = 25cm	1 Stiffener (t=23cm)	2 Stiffener (t=23cm)
x (m)	3.25	3.25	3.25	3.25
y (m)	3.1	3.2	3.2	3.2
t (s)	0.0086	0.0078	0.0082	0.0082
θ_p (°)	37.4205	35.398	37.254	38.0085
σ_{max}	1.83205	1.59372	1.73695	1.68116

For all types of stresses evaluated, the model with an additional 2 cm of slab thickness provides the greatest reduction in stresses. The percentage of reductions in stresses from adding 2 floor beams are close to the values of an additional 2 cm of slab thickness, but overall adding slab thickess remains the more efficient option. All models reveal that the locations of maximum principle stresses are the same, but the times at which they occur vary slightly.

5.Conclusions

Based on the results of analysis of the plates loaded with localized blast loads, there are several points that can be concluded. The largest dynamic response of the structure takes place during the free vibration phase, not the positive or negative phase. This point is proven by the evaluation of the slab deflections for all cases observed. Although the load is no longer effectively acting on the structure, the initial conditions from the previous phases are able to create deflections larger than when the load is actively acting on the slab.

Generally, adding to the slab thickness is more effective than adding floor beams. By adding 2 cm to the slab thickness, the maximum principle stresses are reduced by 12%, whereas adding 1 and 2 floor beams, the reduction in maximum principle stresses are 4.8% and 10.1% respectively. By adding 2 cm to the slab thickness, the minimum principle stresses are reduced by 12%, whereas adding 1 and 2 floor beams, the reduction in minimum principle stresses are 4.8% and 10.2% respectively. These figures show that reduction in maximum and minimum principles stresses are nearly identical. By adding 2 cm to the slab thickness, the maximum shear stresses are reduced by 13%, whereas adding 1 and 2 floor beams, the reduction in maximum shear stresses are 5.2% and 8.2% respectively. Adding 2 cm to the slab thickness reduces deflections at midspan by 16.65% and adding 2 floor beams will only reduce deflections by 13.1%.

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