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Interaction between Concrete Cladding Panels and Fixings under Blast Loading

Y. G. Pan & A. J. Watson

Department of Civil and Structural Engineering, University of Sheffield, Sheffield S1 3JD, UK

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Abstract

This paper reports the interaction between concrete cladding panels and fixings under explosive blast loading. A dynamic finite element analysis was carried out using DYNA3D. Generally, the natural period of vibration of normal size cladding panels is much longer than the duration of positive blast loading, which dictates that the response of the panel is influenced by the blast impulse. Forces transferred to the fixing are highly dependent on the panel stiffness and the plastic deformation developed in the panel. While the axial force in the fixing varies little with the ductility ratio of the panel, both shear force and bending moment in the fixing increase with increasing ductility ratio. All force components have to be considered in the fixing design. © 1996 Published by Elsevier Science Limited.

Keywords: concrete cladding panel, fixing, blast loading, dynamic finite element analysis, natural period of vibration.

INTRODUCTION

The majority of precast concrete cladding panels are designed to withstand wind load and construction or erection loads and should not be subjected to stresses due to structural movement of the frame. This requires that the cladding panel should be isolated from the main structure.

If a building is exposed to explosive blast pressure, then under such dynamic loading, the response of the cladding panel depends not only on the panel parameters, but also on the fixings as failure of fixings will limit the full potential of the panel to be utilised. However, in this paper the failure is restrained in the panel by assuming the fixing to be elastic so that no failure of fixing will interact with the panel response. The fixings can then be designed properly based on these force components or assessed considering the actual fixing type and resistance. There are many investigations of panel response,[1–4] but few on the fixing response.

The analysis of a single degree of freedom (SDOF) model of a cladding panel can give reasonable predictions of flexural response under dynamic loading as long as the fixings work as expected, but recent investigations have demonstrated that the SDOF method is not suitable for predicting the support reactions.[5,6] This is unfortunate as a knowledge of the support reactions is important for fixing design. Even more difficult is to assess the residual capacity of fixings after the panel has been subjected to blast loading since this often demands extensive and costly dismantling of the cladding panels. In a study of cladding response to blast loading, the cladding can be considered as a structural assembly including the panel and the fixings and it is very important to know how the blast forces are distributed and transmitted from the panel to the fixing and how the blast energy is dissipated by panel deformation. The fixing itself can be modelled as a mass-spring system as described by Pinelli *et al.*[7]

REINFORCED CONCRETE CLADDING PANELS

Many types of cladding panel are storey height and span vertically. Some panels employ vertical strengthening 'ribs' so that the panel 'web' then acts as a slab spanning horizontally between these ribs. As a general guide, storey height panels in the order of 4 m span are usually 150–180 mm thick,[8] or use a span/web thickness ratio of 27.[9]

There are two main types of fixings for pre-cast concrete cladding: those which are loadbearing and those which simply offer restraint. Loadbearing fixings are designed to transfer the weight of the cladding panels to the building structure and are usually designed as pin-jointed supports.[8] Restraint fixings are designed to hold panels back to the structure and transfer all horizontal forces (such as wind pressure or suction) to the structure. These fixings need to be designed to accommodate any differential movement between structure and cladding. In the case of average-sized units, there are usually four restraint fixings per panel, located as close as possible to the corners of the panel, with slotted connections which can be used to adjust the relative position between the panel and the fixings and to adjust the position of the panel during erection. The loadbearing fixings do not generally contribute to the lateral resistance under horizontal loading. Typical restraint and loadbearing fixings are shown in Fig. 1.

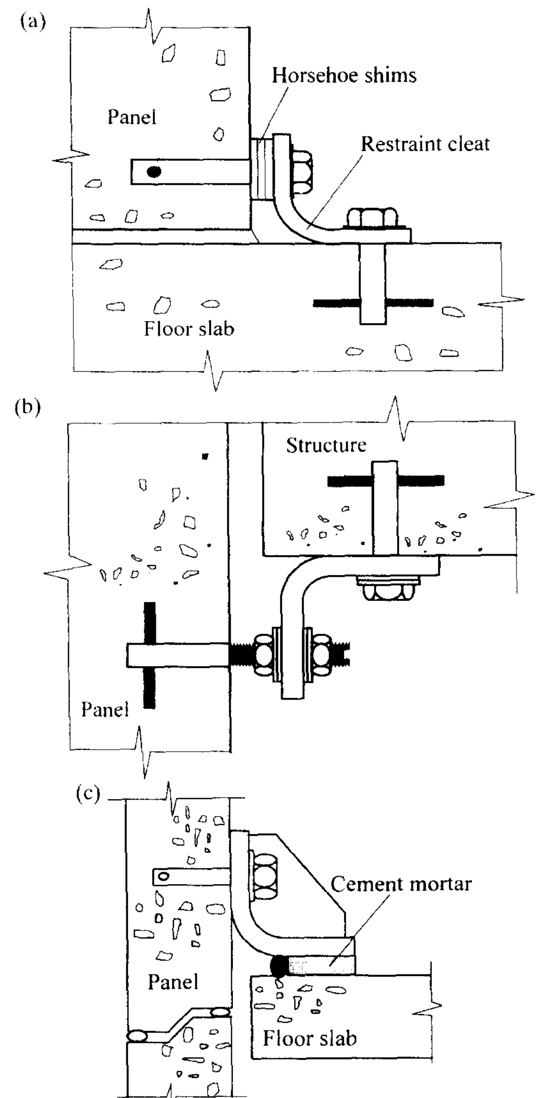

Fig. 1. Typical drawings of fixings for reinforced concrete cladding panels: (a) typical restraint fixing at bottom of panel, (b) typical restraint fixing at head of panel, and (c) typical loadbearing fixing.

BLAST LOADING

An explosion causes an air shock which produces a wave of almost instantaneous increase in pressure above ambient atmospheric pressure on structures in the path of the wave. This is commonly referred to as overpressure, and the initial peak overpressure in air falls rapidly with distance from the explosion and is followed by a phase when the pressure is below atmospheric — the negative phase. The negative phase is of longer duration and of lower intensity than the positive phase.[10,11] Generally, the characteristics which define a blast wave are arrival time t_a, positive duration t_d, peak overpressure p_o, decay coefficient α, negative duration t_{d1}, and peak underpressure p_{o1}. In many cases, the effect of the negative phase is ignored, thus leaving four characteristics t_a, t_d, p_o, and α to define the blast wave (Fig. 2). The blast impulse i which affects the structural response in the impulsive loading range, can be obtained from those basic parameters by integrating the blast overpressure over the period of positive duration.

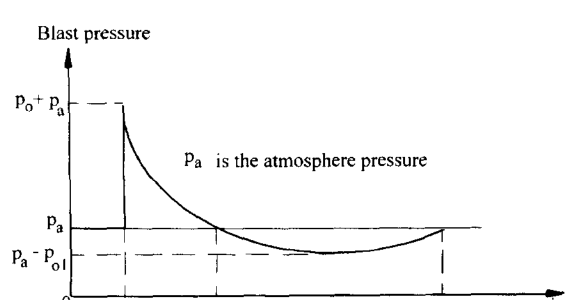

Fig. 2. Typical blast pressure in air at a fixed distance from the explosion.

The decay of blast overpressure is usually described using the modified Friedlander equation.[11]

$$p = p_o\left(1 - \frac{t}{t_d}\right) e^{-\alpha(t/t_d)} \quad (1)$$

where

p_o is the peak reflected overpressure;
t is time measured from the arrival of the blast wave;
t_d is the positive duration of the blast wave;
α is the decay coefficient.

When the shock wave reaches a structure, reflection occurs and the blast loading on the building is measured as the normal component of the reflected overpressure. To give some idea of the magnitude of the blast overpressure, Fig. 3 shows two cases of blast overpressure reflected from a surface normal to the blast wave generated by two charge sizes of 10 kg and 100 kg TNT, respectively at a stand-off of 10 m. The load curve 1 produces an impulse of 283 Pa·s.

It has been found that the effect of a blast wave on a structure is greatly influenced by the ratio of the positive duration of the air shock and the natural period of vibration of the element of the structure under consideration. If this ratio is less that 0.1, then the specific impulse, the time integral of pressure, is the dominate characteristic of the load, and the loaded area of the structure acquires a velocity in an extremely small displacement.[12,13] If this ratio is greater than about 6, it is the peak overpressure that dominates the structural response.[12]

Two programs (BLAST and DYNALO)[14] are used to determine the characteristics associated with free air explosion and the blast loading

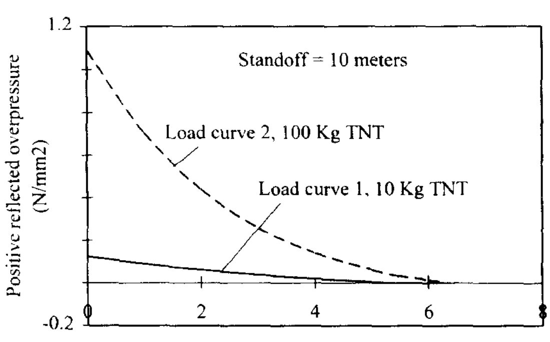

Fig. 3. Blast loading on the face of a building generated by reflection.

imposed on a building surface. To help estimate whether a blast load is in the impulsive loading range or quasi-static loading range, it is useful to know the possible maximum duration of a blast wave generated by a given charge size. Equation (2) is obtained by regression based on results from BLAST and can be used to predict the approximate maximum duration for different charge sizes.

$$t_{d\,max} = 3.965 + 5.524 \log(w) - 1.793 \log^2(w) + 1.388 \log^3(w) \quad (2)$$

where

$t_{d\,max}$ is the maximum possible duration in milliseconds; and
w is the equivalent weight of TNT in killograms which releases the same energy as the actual explosive.

For example, t_{dmax} is about 4 ms for a charge of 1 kg TNT, suggesting that for many building elements no matter what the stand-off might be, it is the impulse that influences the structural response. For a charge of 10 kg TNT, t_{dmax} is about 9 ms with a large stand-off compared with $t_d = 5.6$ ms when the stand-off is 10 m as shown in Fig. 3. This kind of information would be useful when planning a test or carrying out post-incident assessment.

NUMERICAL MODELLING

Different cladding fixings of the types described in the previous section are used in practice, but the objective of the present work is to see how blast pressures on the cladding are transferred to forces in the fixings and how the variation of force with time depends upon the relationship between the cladding panel and the fixing properties. By assuming that the fixings are elastic, the maximum forces that could be transferred to fixings can be analysed. These data can then be compared with load capacities of actual fixings in a later stage. The cladding panel is assumed to be square and supported at four corners. Figure 4(a) shows a quarter of the panel.

A dynamic finite element analysis program, DYNA3D, is used in the study, and the concrete slab is modelled using a total of 144 shell elements (12 × 12). Five Gauss points are required for integration over the cross section. The fixing is assumed to be a rectangular bar of

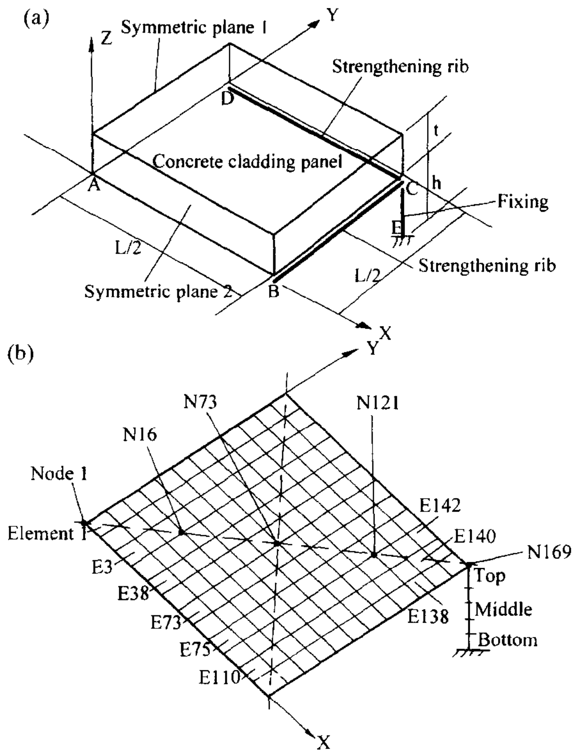

Fig. 4. Modelling of a quarter of the cladding assembly: (a) a quarter of the cladding assembly, and (b) node and element numbering.

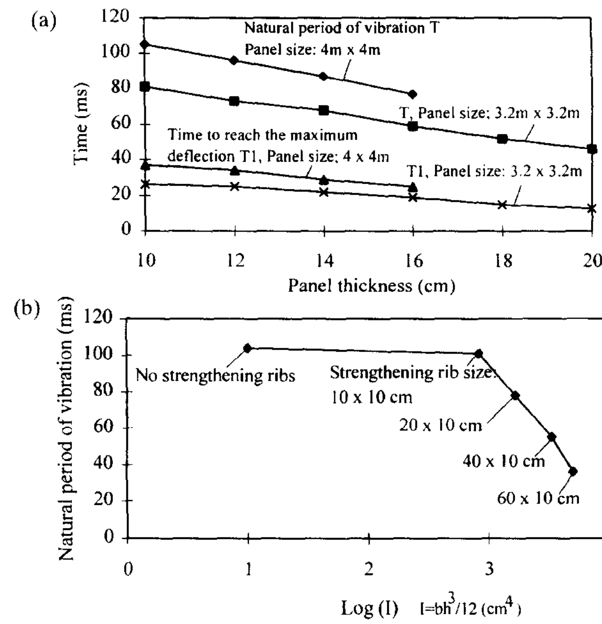

Fig. 5. Effect of panel thickness on the natural period of vibration: (a) natural period of vibration of cladding panels, and (b) effect of stiffness of strengthening ribs.

2×2 cm in cross section and is modelled using five beam elements along the length of the fixing (Fig. 4(b)).

At this stage the fixings are assumed to be linear elastic and made of steel with $E = 200$ GPa, $v = 0.3$. This makes the interpretation of DYNA3D results easier as only failure of the panel is considered. The strength of fixings will be considered in the next phase of our project. The reinforced concrete cladding panel is simplified as an ideal elastic plastic material having a plastic moment equal to the ultimate bending moment of the actual panel. The modulus is assumed to be the same as that of plain concrete. Other properties are the density of reinforced concrete $\rho = 2400$ kg/m^3, and Poisson's ratio $v = 0.2$.

In order to see whether the panel is influenced by the impulse or the peak overpressure, the natural period of vibration of the panel, T, is determined first. The vibration is initiated by applying a uniformly distributed blast loading on the panel, and the fundamental natural period of vibration is determined from the deflection history of the central point (Node 1) of the panel. The panel thickness and natural period are linearly related and the thicker the panel, the smaller the natural period. The varying range of the natural period, T, for a panel of 3.2×3.2 m in size is shown from the DYNA3D analysis to be about 80–45 ms when the thickness is increased from 10 to 20 cm (Fig. 5(a)). If these panels are subjected to blast load curve 1 in Fig. 3, t_d/T varies from 0.07 to 0.12, and it can be seen that it is the impulse of the blast wave that affects the panel response. Increasing the panel size to 4×4 m, the natural period of vibration is increased by 30%, resulting in the ratio of $t_d : T$ being even smaller.

The stiffness of the strengthening ribs affects the natural period of the panel (Fig. 5(b)), and the effect is dependent on the dimensions of the strengthening ribs. For strengthening ribs with a size of 10×10 cm the reduction in the natural period of vibration is less than 10% and the panel is still in the impulse range, but when the ribs are 60×10 cm the reduction in the natural period is much greater.

RESPONSE OF THE CLADDING PANEL

A cladding panel of 3.2×3.2 m in size and 14 cm in thickness without strengthening ribs subjected to the load curve 1 ($i = 283$ Pa·s) shown in Fig. 3 is defined as the reference case.

The deflection of the panel continues after the blast loading has decayed to zero. Depending on the natural period of vibration of the panel, the time to reach the maximum deflection varies. For a given blast duration the longer the natural period, the longer it takes to reach the maximum deflection (Fig. 5(a)). Based on the bending moment history in the central element, an impulse of 90 Pa·s is determined as the elastic limit. With increasing impulse the bending moment increases as well until full plastic moment is reached. Figure 6(a) shows the deflection history at a few selected points on the panel at the elastic limit, and Fig. 6(b), the deflection history of the central node (Node 1) under greater impulses. Under all loading conditions the deflection at the supporting point (Node 169) is nearly zero so the total deflection at the central point is the net deflection. It can be observed based on Fig. 6(a) that the time to reach the maximum deflection varies depending on the node position on the panel, and the maximum difference is about 3 ms. Neither do they return to the zero deflection position at the same time, but the time difference is just about 1 ms. Based on results in Fig. 6(b), it can be seen that the higher the impulse, the longer it takes to reach the maximum deflection. It can be noticed that the increase in load capacity from the elastic limit can be much greater than that in a static test as long as significant plastic deformation is allowed. For example, the increase in impulse from the elastic limit of 90 to 600 Pa·s is over

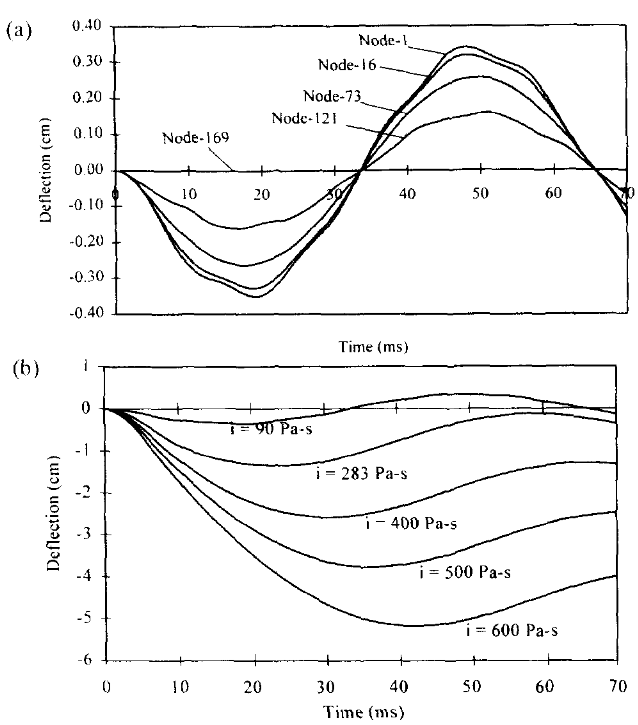

Fig. 6. Deflection history of the panel under the impulse: (a) deflection history under the impulse of 90 Pa·s, and (b) deflection history at the centre of the panel under different impulse.

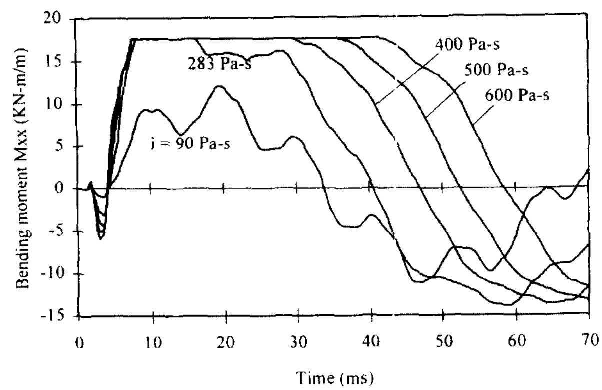

Fig. 7. Bending moment history in element 1 under different impulse.

550% if the panel can accommodate a deflection of about 50 mm. If a larger deflection is allowed, an even higher increase in impulse can still be expected. In contrast to this, a plastic limit is reached when a mechanism is formed no matter what the deflection might be. After the whole blast energy is absorbed the panel starts to move backwards and enters into the era of free vibration.

In addition to the deflection history, we are also interested in the stress/force distribution within the panel. Figure 7 shows the history of bending moment M_x, of the central element (Element 1) under different impulses. It can be seen that once in the plastic range the bending moment remains constant until the maximum deflection is reached and the panel starts to move backward. Figure 8 shows the moment history of three corner elements under an impulse of 283 Pa·s. It can be seen that during the first few milliseconds the bending moment along the symmetric plane is very small whilst the moment around the support position experiences a sharp increase. After the first few milliseconds, the bending moment along the symmetric plane becomes greater than that at

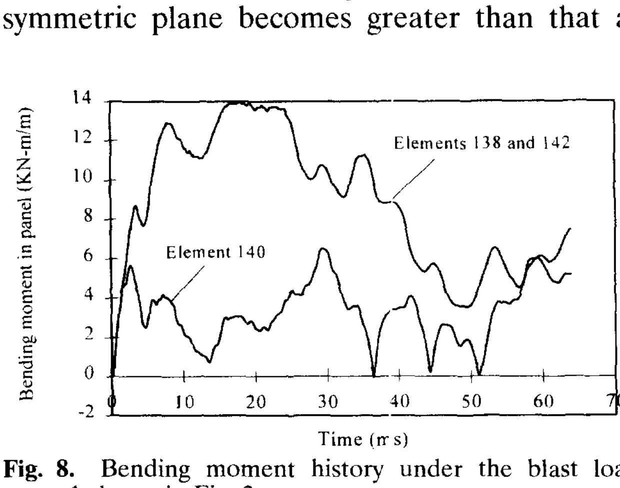

Fig. 8. Bending moment history under the blast load curve 1 shown in Fig. 3.

the corner. The maximum moment at the centre of the panel is then much higher than that around the corner area. While the deflection curve follows a sine curve quite well, the moment history curve clearly shows multi-degree vibration.

Increasing the thickness of the panel reduces the maximum deflection for a given blast load, but the relative decrease in deflection is smaller than that which can be achieved if the panel is statically loaded. The former generally follows the square inverse of the thickness ratio, and the latter the cube inverse of the thickness ratio. Generally, increasing the panel thickness enhances the elastic load capacity. For reinforced concrete cladding panels, however, the relationship is more complicated. In a static case we can have two cross sections having the same bending capacity, one using a thicker panel and less reinforcement, and the other a thinner panel and more reinforcement. But when these two panels are subjected to blast loading, the resistance of the first panel is less than that of the second one because the resistance is proportional to the product of panel thickness and equivalent yield strength and the latter increases at a higher rate than the decreasing rate of panel thickness. Though increasing the thickness increases the static capacity generally, the equivalent yield strength of the composite slab which is obtained by equating the full plastic bending moment of the assumed elastic–plastic material to the ultimate bending capacity of the RC panel will be reduced. This results in an increase in blast resistance which is much smaller than that which can be expected in a static case. On condition that the equivalent yield strength remains unchanged, the increase in elastic load capacity to blast loading is enhanced linearly with the increase in panel thickness, whilst the static capacity is enhanced with the square of the thickness. This means that if the panel thickness is increased twice, the increase in blast resistance of the panel is 100% while the increase in static capacity is 300% (Fig. 9).

If plastic deformation is allowed to develop to resist external load (curves corresponding to $i = 283$ Pa·s and above), the resistance to blast loading is the product of the resistance at the elastic limit and a function of the ductility ratio which is defined as the ratio of plastic deflection to the deflection at the elastic limit, but the resistance to static loading is limited by the formation of a mechanism in a structure. For example, the resistance of a simply supported rectangular beam to a uniformly distributed static load is increased by a certain amount from the elastic limit to plastic limit when a plastic hinge forms in the centre of the beam, 50% for a beam made of an elastic plastic material and other values for a reinforced concrete beam depending on reinforcement details. Assuming the panel fails at a given ductility ratio, the significance of increasing the resistance at the elastic limit is obviously very important as it directly affects the final resistance of the cladding panel (Fig. 10).

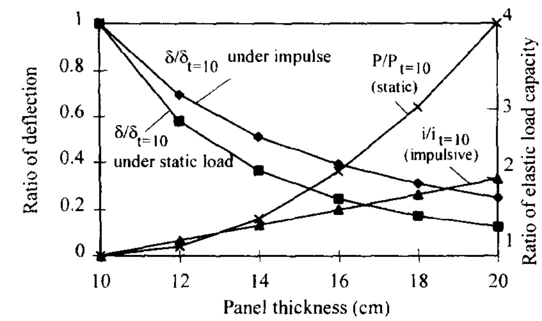

Fig. 9. Effect of panel thickness on deflection and load capacity in the elastic range.

RESPONSE OF FIXINGS

To decide whether a fixing is working as expected, the prerequisite is to know what level of forces will be transferred to the fixings. These include the axial force, shear force, and bending moment. These force components will be affected by fixing stiffness, but in this paper only one fixing stiffness is considered and the attention is focused on how these force components develop with increasing levels of blast

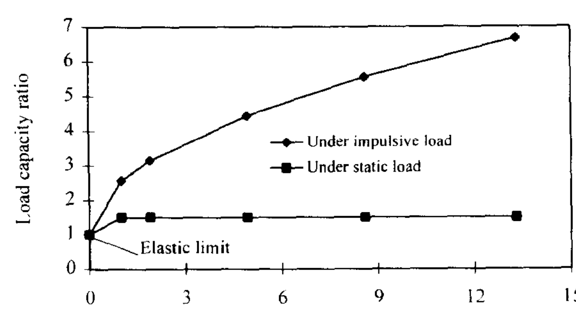

Fig. 10. Effect of ductility ratio on the load resistance in different loading range.

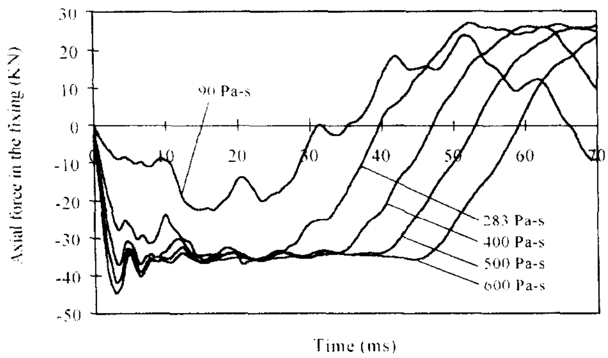

Fig. 11. Axial force history in the fixing.

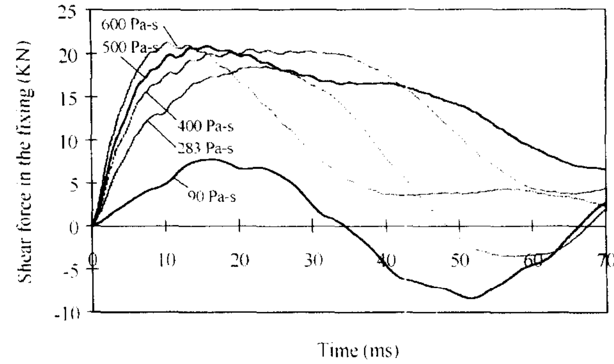

Fig. 12. Shear force history in the fixing.

loading. When the panel is in the elastic range there are three stages of axial force development in the fixing before the maximum axial force is reached, e.g. for an impulse of 90 Pa·s in Fig. 11: (i) a fast rising period when the panel is first hit by the blast wave, (ii) a fairly slow variation period for 3–5 ms, (iii) another fast rising period until the maximum force is reached at about 15 ms. If the blast loading causes plastic deformation in the panel, the axial force experiences a faster rising period then drops slightly to a stable level until the panel moves backward and changes gradually from compressive to tensile force. The magnitude of the peak axial compressive force increases with increasing impulse and can be 25% higher than that at the stable level. In contrast, the magnitude of the axial force at the stable level changes little with increasing impulse.

Though shear forces transferred to the fixing have about the same maximum value when different degrees of plastic deformation occur in the panel under different levels of blast loading, it is difficult to tell when the plastic deformation starts or when it ends based on shear force history curves. The increase in the maximum shear force from elastic limit corresponding to $i = 90$ Pa·s to the states when plastic deformation occurs ($i \geq 283$ Pa·s) is more than that in the axial force, 175% compared with 65%. This is because, along with the increase in the deflection, the fixing is forced to move further towards the centre of the panel resulting in the shear force being increased. When the impulse is increased from 90 (corresponding to the elastic limit) to 283 Pa·s, both the maximum shear force and the time to reach the maximum value are increased, from about 7 to 18 kN and from 15 to 23 ms, respectively. However, with further increase in blast impulse, the shear force tends to reach its peak value earlier, for example 18 ms at $i = 400$ Pa·s, and only 11 ms at $i = 600$ Pa·s (Fig. 12).

The bending moment in the fixing varies significantly along the length in terms of both magnitude and pattern (Fig. 13). At the elastic limit which corresponds to the input impulse of $i = 90$ Pa·s, the top part of the fixing experiences a certain level of bending moment (0.4 kN·m), the moment at the middle part is about half the value of that at the top and the moment at the bottom of the fixing is nearly zero. When the impulse is strong enough to cause plastic deformation in the panel, the bending moment in both the middle and bottom part of the fixing increases monotonically with increasing impulse, but the maximum value of moment at the top part is reaching an asymptotic value of 1.5 kN·m when the blast impulse is greater than 500 Pa·s. This level of bending moment is very large for a fixing made from a single bar, for example, assuming a yield strength of 250 N/mm^2 the full plastic bending capacity of the fixing is 0.5 kN·m. When the effect of axial force is considered the bending capacity will be reduced indicating the fixing may reach its capacity before the panel develops maximum possible plastic deformation. If the fixing fails due to formation of a plastic hinge, the panel can still undertake more impulsive load by developing further plastic deformation. However, if the fixing fails due to crushing of concrete at the connection position, the cladding fixing assembly will lose its capacity completely.

DISCUSSION

Because inertia forces are involved in the analysis of the panel and fixings under blast loading,

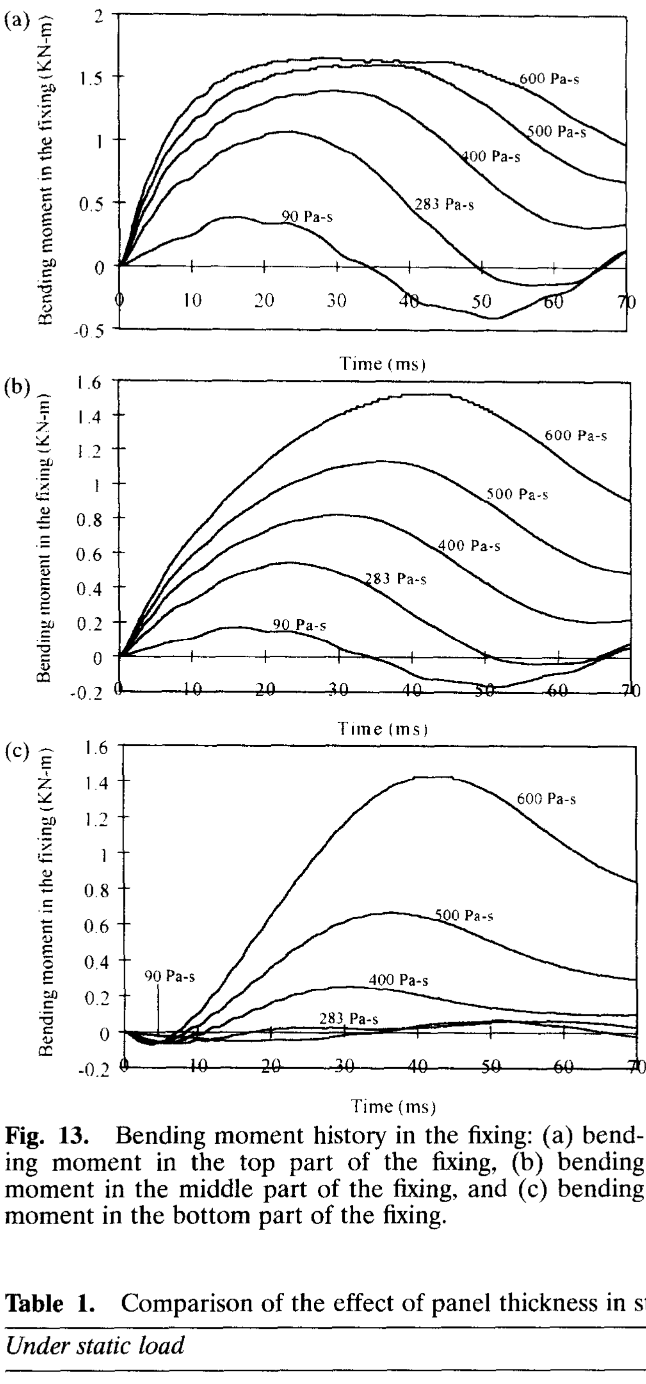

Fig. 13. Bending moment history in the fixing: (a) bending moment in the top part of the fixing, (b) bending moment in the middle part of the fixing, and (c) bending moment in the bottom part of the fixing.

some traditional concepts for static mechanics cannot be applied in a dynamic analysis to blast loading.

For instance, increasing the panel thickness will reduce the panel deflection, but the decrease is less than that can be expected in a static test. In the statically determinate case, the external bending moment is not dependent on the panel thickness ignoring the effect of self weight. Table 1 demonstrates the effect of panel thickness on deflection and load capacity in the static and impulsive loading range.

It shows that for a static case the maximum deflection is reduced following the cube inverse of the thickness ratio, but under impulsive loading the reduction follows the square inverse of the thickness ratio.[14] Checking the relationship between moment and thickness, it can be found that under a static loading the moment is independent of panel thickness. However, under the impulsive loading an increase in panel thickness causes an increase in bending moment, which means that part of the contributions of increasing panel thickness are cancelled by the increase in bending moment. This is reflected in the relationship between the load capacity and panel thickness. Assuming the panel fails when the maximum stress in the panel reaches a certain level, the load capacity-thickness relationships (P–h for static and i–h for impulsive) suggest that the static load capacity increases with the square of the thickness, but the impulsive load capacity increases only linearly with the thickness. Considering changes in load capacity, the deflection–thickness relationships are similar in both static and impulsive loading ranges, i.e. the maximum deflection at

Table 1. Comparison of the effect of panel thickness in static and impulsive loading cases

Under static load	*Under impulsive load*
$w_o = \gamma_{s1}\dfrac{P}{h^3}$	$w_o = \gamma_{i1}\dfrac{i}{h^2}$
Using the relationship $M = EI\dfrac{\partial^2 w}{\partial x^2} = EI\omega_o\dfrac{\partial^2 f(x,y)}{\partial x^2}$ and $I = \dfrac{bh^3}{12}$ we can obtain:	
$M = \gamma_{s2}P$	$M = \gamma_{i2}\cdot i\cdot h$
Knowing that the maximum stress $\dfrac{My_{max}}{I}$ we can relate the load to the maximum stress	
$P = \gamma_{s3}\sigma h^2$	$i = \gamma_{i3}\sigma h$
Substituting in the expression for the maximum deflection w_o	
$w_o = \gamma_{s4}\dfrac{\sigma}{h}$	$w_o = \gamma_{i4}\dfrac{\sigma}{h}$

the load limit varies with the inverse of the panel thickness. Because these relationships apply only to elastic materials, the findings are limited within the elastic range. The change in natural period which results from a change in thickness will not alter the response of the panel provided the ratio of t_d/T is still less than 0.1.

From DYNA3D results the increase in bending moment is about 20% in the central area and about 30% in the corner area when the panel thickness is increased from 14 to 18 cm, an increase of 29%. The increase in bending moment in the central area is slightly lower than the relative increase in panel thickness. This is probably because the panel does not fully comply with the condition in the impulsive loading range and the characteristics under other loading ranges may contribute to this variation.

The blast resistance of a cladding panel depends on the capacity of the panel to develop plastic deformation. This capacity can be defined by the ductility ratio. The higher the ductility ratio the higher the capacity in plastic deformation and the greater the energy absorption, hence the higher the blast resistance.

The existence of a strengthening rib on the panel can increase slightly the bending resistance around the edge and reduce the overall deflection of the panel. But for a large panel of 3 m wide and 4 m long, the effect of 10×10 cm concrete strengthening ribs is not significant.

When the panel develops plastic deformation, the axial forces transferred to the fixing change little with increasing ductility ratio, but both the shear force and the bending moment in the fixing increase with an increasing ductility ratio. It is clear that all three force elements have to be considered in the fixing design.

CONCLUSIONS

Dynamic finite element analyses were carried out to see the interaction between cladding panel and fixings under explosive blast loading. The following conclusions are drawn:

(i) Generally, the effect of the stiffness of the fixing on the panel response is very small in the normal range of fixing sizes, but the panel stiffness has a profound effect on both the forces transferred to the fixing and on the panel response itself.

(ii) Increasing the panel thickness will reduce the deflection, but the decrease is less significant than in a static test. The bending moments in the panel will also be increased in proportion to the panel thickness. This will make the increase in load capacity less significant than that in a static situation.

(iii) The inclusion of strengthening ribs will not change the panel and fixing response except for the bending moment in the central area which will be reduced slightly. The significance is that the corner area becomes safer.

(iv) All components of the forces transferred to the fixings need to be considered in the fixing design. During the first few milliseconds after the arrival of the air shock the fixing is mainly axially loaded. As flexural deformations of the panel develop, shear and bending moments transferred to the fixing are gradually increased.

REFERENCES

1. Luckyram, J., Stewart, B. M. & Zintilis, G. M., Blast loaded R. C. slabs: a binary response model for shear and flexural interaction. *Proc. of the 2nd Int. Conf. on Structures under Shock and Impact*, Portsmouth, UK 16–18 June 1992, pp. 177–188.
2. Sheridan, A. J. & Cowdery, C. A., An analysis of shear/flexure coupling applied to the failure of reinforced concrete structures. *Proc. of the 2nd Int. Conf. on Structures under Shock and Impact*, Portsmouth, UK 16–18 June 1992, pp. 189–197.
3. Crouch, R. S. & Chrisp, T. M., The response of reinforced concrete slabs to non-nuclear blast loading. *Proc. of the 1st Int. Conf. on Structures under Shock and Impact*, Cambridge, Massachusetts, USA, July 1989, pp. 69–76.
4. Hulton, F. G., The behaviour of steel-backed reinforced concrete panels under explosive attack. *Proc. of the 7th Int. Symposium on Interaction of the Effects of Munitions with Structures*, Mannheim, Germany, April 24–28 1995b, pp. 111–116.
5. Hoiseth, K. & Weerheijm, J., Structural concrete slabs under impulsive loads. *Fortifikatorisk Natat Nr 211/93*, Norwegian Defence Construction Service, edited by T. Krauthammer, 1990, pp.125–148.
6. van Wees, R. M. M. & Peters, D.-J., Accuracy and range of applicability of the single-degree-of-freedom method. *Proc. of the 7th Int. Symposium on Interaction of the Effects of Munitions with Structures*, Mannheim, Germany, 24–28 April 1995, pp. 371–380.
7. Pinelli, J.-P., Craig, J. I. & Goodno, B. J., Energy-based seismic design of ductile cladding systems. *J. of Struc. Eng.*, **121** (3) (1995). 567–578.
8. Architectural Cladding Association, *A Guide to the Design, Detailing and Economics of Precast Concrete Claddings*, C1/SfB/ /41/Gf/ / 1990.
9. Brookes, A.J., *Claddings of Buildings*. Construction Press, London, 1983.
10. Kinney, G.F. & Graham, K.J., *Explosive Shocks in Air*, 2nd edition. Springer-Verlag, New York, 1985.
11. Baker, W.E., *Explosions in Air*. University of Texas Press, Austin, Texas, 1973.

12. Baker, W.E. *et al.*, *Explosion Hazards and Evaluation*. Elsevier Scientific Publishing Company, Oxford, 1983.
13. Watson, A.J., The response of civil engineering structures to impulsive loads. *Proc. of the 3rd Int. Conf. on Structures under Shock and Impact*, Madrid, Spain 1–3 June 1994, pp. 3–10.
14. Pan, Y. G., Watson A. J. & Hobbs, B., *Vulnerability Assessment of Building Cladding Systems to Blast Loading*, (progress report 10 July 1995). Department of Civil and Structural Engineering, University of Sheffield, May–July 1995.