Dealing with blast loading on brickwork

Synopsis

Much effort is being deployed in maintaining electricity generation from the UK's fleet of nuclear power stations with safety being a prime consideration. Up-to-date safety standards demand resistance to hazards which normally include seismic and blast loading, yet many stations were built before such considerations became mandatory. To extend the generating life of older nuclear power stations, a demonstration of conformity to modern safety standards is now required.

This paper will discuss the structural problems associated with assessing and strengthening masonry panels subjected to blast loadings as generated by a postulated accident of hot gas release from fracture of a reactor pressure vessel's cooling circuit.

The paper will provide background on masonry strength and assessment techniques, including the use of dynamic amplification which permits static design principles to be used under dynamic loading conditions. An account of the strengthening options and the problems associated with their implementation will be presented.

Notation

DAF	Dynamic Amplification Factor
E_{static}	Static elastic modulus of masonry
$E_{dynamic}$	Dynamic elastic modulus of masonry
F	Frequency of structure
H	Height of masonry panel
L	Length of masonry panel
M	Applied bending moment
P_{lat}	Applied lateral pressure
r	Arch rise
t	Thickness of masonry leaf
Z	Elastic modulus of masonry in direction of bending
f_k	Masonry characteristic design stress in compression
$f_{kx}II$	Masonry characteristic design stress in flexure (failure plane parallel to bedjoints)
$f_{kx}L$	Masonry characteristic design stress in flexure (failure plane perpendicular to bedjoints)
δ	Arch deflection
λ^2	Mode shape factor
γ	Mass per unit area of masonry panel
γ_m	Material partial safety factor
γ_f	Load partial safety factor
μ	Poisson's ratio of masonry

Introduction

The UK has a significant fleet of nuclear power stations which currently provide around 20% of the nation's electricity. Decommissioning of many early reactors is underway but for others, favourable economics, combined with the need to maintain base load capacity, has made life extension desirable.

To extend a generating licence, an operator must demonstrate his system conforms to modern standards. In the case of nuclear stations this necessitates structural qualification for seismic events and qualification against a range of accidental loadings that may be derived from postulated plant failures. Of these, rupture of pressurised pipe work in the reactor cooling circuit is one of the more severe. This event would lead to pressurisation of the building's internal compartments and potential failure of safety critical components.

The design target after such an accident is to vent gas along defined routes permitting the plant to be safely shut down. But this is not an easy task since the pressure circuit runs at over 10 atmospheres with escaping gas at several hundred degrees, thus the heat exchange medium (in this case CO_2) escapes with blast force. The studies associated with such failures are known generically as 'hot gas release' studies.

The heat transfer medium from reactor to boiler is in this case pressurised CO_2 gas carried along pipe work designed to high levels of integrity. The safety case seeks to assure the integrity of such circuits against every conceivable operating condition. But it also considers the possibilities of failure and seeks to assure that the failure effects are mitigated, always with the target of safe reactor shut down. In the buildings under consideration, a number of pipe failure locations were identified and then routes defined along which gas had to flow prior to its vent to atmosphere. Structures along this route had to be qualified to contain the gas in the manner of a duct.

Reactor buildings dating from the 1960s often have structures made up of substantial concrete frames including columns, slabs and beams. Unfortunately they also often have brickwork panels forming the dividing walls. In hot gas release projects such masonry panels lining the duct route must sustain the blast and gas pressure loadings. This paper will describe the assessment and strengthening of masonry elements to sustain the extraordinary forces involved.

The brickwork panels

The masonry panels considered for assessment were all division walls within older nuclear power stations. As such, they were generally not designed for any specific lateral load and certainly not dynamically rated. The panels were often constructed from common bricks produced by the London Brick Company (now part of Hanson). Several important factors must be stated concerning the brickwork's physical construction:

- the panels are of two stretcher bonded leaves with no cross bonding in the 10mm cavity, giving 215mm thickness overall,
- some panels contain cross-cavity mesh reinforcement but no cavity fill mortar,
- the orientation of the brick frog is not uniform but generally bricks were laid frog-down,
- there are no ties between the masonry panel and surrounding building frame.

The masonry panels are therefore in possibly the worst structural condition they could be to resist lateral load. Moreover their capability is modified by the attachment of many services, which are often of significant weight. Some of these services are 'safety related', so from that standpoint alone, the panels cannot be allowed to fail under hot gas release loading. A robust design requires them to survive.

Dynamic loading

In general any loading may be categorised as static, quasi-static or dynamic, the difference depending on the duration of load application and the corresponding structural response. So all types of 'dynamic loading' are associated with variations of load intensity with time. If the loading is very rapid or impulsive, the structure simply has no time to respond and the instantaneous application of high impulsive load may be of little significance. On the other hand, in time varying motion such as seismic loading where the period of the motion is similar to the structural period, the structure may effectively experi-

B. Jonathan Haynes
B.Eng(Hons), MSc, PgCertHE, C.Eng, FIStructE, MICE, FHEA
University of Salford (formerly Jacobs)

M. Faisal Beg
B.Eng(Hons)
Mouchel Parkman (formerly Jacobs)

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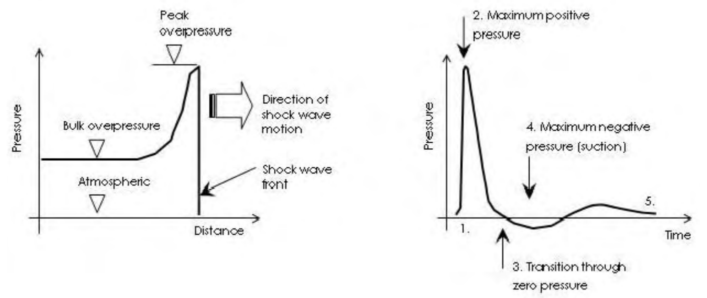

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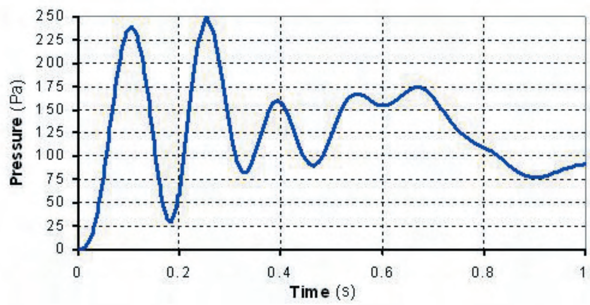

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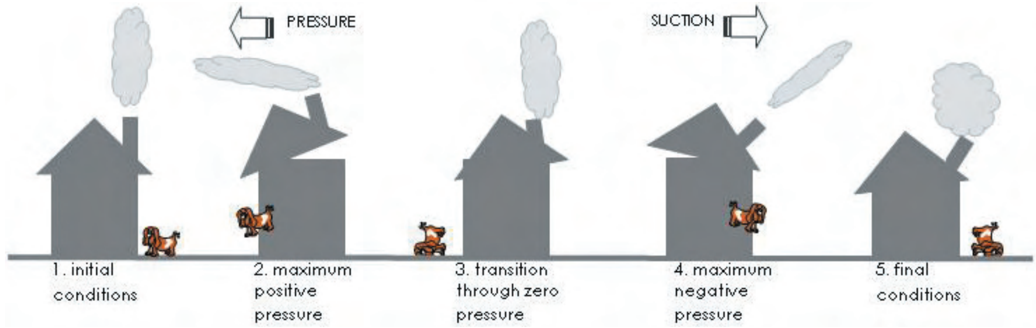


ence amplification of the applied loading. The interdependence between the nature of the applied loading and the structure is the subject of dynamic analysis. Key parameters are duration of the load application and the relationship between the natural frequency of the structure and the frequency (or duration) of the input motion. Classically, and in the extreme, if the frequency of input motion equals the natural frequency of the structure, a condition of resonance exists and the dynamic amplification of the loading is potentially huge, limited only by damping. Blast loading is of a dynamic type described by a pressure impulse (i.e. very short time of application) but it is non-periodic. Generally the loading period is much shorter than the structure's period. To derive an equivalent static loading there must be knowledge of the pressure – time loading history and knowledge of the responding structure's dynamic characteristics.

Blast loadings are highly variable in nature and much of our subject knowledge stems from theory over a century old combined with experimental back up from the nuclear weapons testing programme conducted during the 1950s[1]. When explosives detonate, a near instantaneous expansion of gaseous material occurs which generates a shock wave. Damage due to explosions is largely due to the incidence of this shock wave on the structure. This wave moves out from the explosion source with a very rapid increase in pressure. This is then followed by a drop to a pressure known as the bulk overpressure. Thereafter, the blast pressure drops below the background pressure so that at some distance behind the shock wave there is a suction phase, this is depicted in Fig 1. Fig 2 depicts the effects of suction passing a point. In open air blasts caused by explosives, the shock wave radiates out hemispherically from the source and the pressure dissipates with distance. But within buildings or in confined areas such as streets, wave reflection can occur which causes local amplification.

The mechanics of gas explosion are very similar to those of explosive chemical blast but the explosive period is much longer and the pressure increases are generally confined within a structure. Consequently the effects of drag and turbulence from pressure waves moving over structural obstacles are much more important but also more complex to assess. For example, reflection of blast waves can significantly increase the effective pressure where incident and reflected waves interact.

In hot gas release studies, the postulated blast loads impart lateral pressure on the brick panels that line vent routes. Unfortunately such pressures act across the panel's weakest axis, creating flexure. Moreover, brickwork is essentially brittle and as such appears a naturally poor medium to resist the violent shock of a blast. Little test evidence exists to provide information on brickwork's ability to sustain such loads. However, following the Ronan Point disaster in the1960s, the British Ceramic Research Association[2] undertook limited gas explosion testing on several arrangements of brickwork panels to investigate the capability of masonry to sustain the introduction of 'accidental load' considerations. Their work suggested, perhaps surprisingly, that un-reinforced solid, one brick thick, masonry panels could sustain dynamic blast loads of around 50kN/m^2 without 'collapse' albeit with considerable damage. Compare this to the normal assumption that panels are weak against wind loads of around 2.0kN/m^2. An explanation of the difference stems from the different pulse durations. Blast loads are high but instantaneous and die away quickly. For wind loads, the gust duration is relatively long and may be periodic. Brickwork crack patterns in the blast tests demonstrated a strong correlation with yield-line assumptions normally witnessed in reinforced concrete slab failures.

British Energy[3] commissioned a series of tests at Bristol University Earthquake Engineering Research Centre, comprising seismic tests on solid, one brick thick, vertically spanning masonry panels. These tests demonstrated that such panels could sustain cycling lateral accelerations of up to 1.0g without collapse. Given the mass of wall as approximately 1.9kN/m^2 this is the sort of value that might be expected from experience of wind resistance.

Recently, Du Pont[4] commissioned blast testing of bi-directionally Kevlar® coated masonry panels (the Kevlar® being intended as reinforcement) and these survived blast loadings of up to 175kN/m^2 (50mS to peak loading) without damage.

Hot gas loading

Obviously, undertaking load tests to model or define the loading pulses for hot gas release would not be feasible inside an active nuclear power station. Hence computer models are substituted to generate pressure transients for blast load definition. This involves creation of a simplified model of the reactor building defining its sub-divided space and any links between the spaces (such as ducts, doorways and service risers). Pressure vessel pipe work instantaneous rupture is then simulated at predetermined locations and from this, fluid dynamic principles can be used to generate time-history transients of temperature and pressure within each of the sub-divided spaces. An example of such a pressure transient is shown in Fig 3.

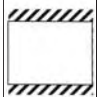
Shock wave and blast overpressure characteristics are described by the

Fig 1. A blast propagation graph and general blast transient/ Fig 2. Effects of the passage of a blast wave / Fig 3. Example of a HOTJET pressure transient

Table 1: Variation in elastic modulus for frog-up and frog-down construction

Value of Elastic Modulus (N/mm^2)	Design vertical compressive strength, f_k	
	(Frog-down) $3.14N/mm^2$	(Frog-up) $5.8N/mm^2$
$E_{static} = 900f_k$	2826	5220
$E_{dynamic} = 270f_k$	848	1566

Table 2: Panel frequencies for various edge restraint conditions

First mode natural frequency f (Hz) $f = \frac{\lambda^2}{2\pi \cdot H^2}\sqrt{\frac{E \cdot t^3}{12\gamma(1-\mu^2)}}$					
Static	Frog-down	6.2	16.3	20.9	15.0
	Frog-up	5.9	15.4	19.8	14.2
Dynamic	Frog-down	3.4	8.9	11.5	8.2
	Frog-up	3.2	8.4	10.9	7.8

L = 5m, H = 3m, t = 0.215m, μ = 0.25, λ^2 = mode shape factor[4], γ = 326 kg/m^2 (frog-down) or γ = 364 kg/m^2 (frog-up)

Striped areas indicates simply supported edge

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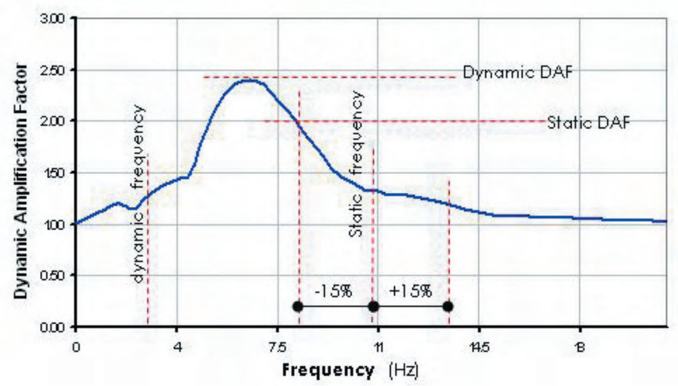


principles of fluid dynamics and may be assessed using Computational Fluid Dynamics software named FLUENT and HOTJET respectively.

The majority of postulated hot gas releases are dynamic in nature, so for this reason it is important to establish the extent of structure response which in turn is dependant upon blast characteristics and structure frequency.

Only static design methods are available for masonry, so to allow for the dynamic loading we introduced the dynamic amplification factor concept, which is the ratio of dynamic to static deflection and is applied in the manner of a load factor on the blast load. The general mathematical solution for the DAF is based upon the Duhamel integral (see Equation 1), this requires a numerical integration of the blast transient, which may conveniently be undertaken using a spreadsheet.

$$\nu(t) = \frac{1}{m\omega}\int_{0}^{t} p(\tau)\cdot e^{-\xi\omega(t-\tau)}\cdot \sin(\omega(t-\tau))\cdot d\tau \qquad ...(1)$$

where $\nu(t)$ is the displacement acting at time t, $p(\tau)$ is the load acting at time $t = \tau$, m is the mass of the panel, ω is the natural circular frequency, which for low damping may be taken as the undamped frequency ($2\pi f$, where f = natural frequency), ξ is the proportion of critical damping.

It is evident from the range of elastic moduli computed, that the frequency of structure response to dynamic loading is not accurately predictable. This is compounded by the fact that in reality boundary conditions will not reflect the analysis assumptions of free, pinned or fixed edges and this is a potential change in stiffness. To complicate matters further, assessment methods being considered relied upon the formation of plastic 'hinges' within the structure whose existence implies significant displacement and hence some further relaxation (loss of stiffness) against the blast load. The presence of plastic hinges automatically affects panel stiffness which in turn affects the DAF.

Natural frequency

The main point about blast loads, as shown in Fig 3, is that the load is applied in a very short period of time, often around 0.1s in the example of hot gas release. Conversely, wind load would be considered as a 3s gust. Because of this, the frequency of the structure must be known to assess if the structure will 'respond' to the loading.

A structure's natural frequency is a key parameter in any dynamic evaluation. If the natural frequency of the masonry panel is close to the peak of the blast frequency, then exaggerated response to loading will occur. Calculation of the natural frequency is therefore of great practical importance but cannot be undertaken with certainty in brick panels for several reasons to do with variable mass and potential variable stiffness (frequency being a function of both mass and stiffness). Firstly, the orientation of brick laying, either frog down or frog up, means that two extremes of panel density must be considered, frog-down brickwork will contain voids and hence have lower mass. Additionally when bricks are laid frog down they have a reduced vertical allowable nominal strength (the brickwork contains voids) and the elastic modulus is derived from this, i.e. the stiffness is also affected. The dynamic modulus is derived from testing by Bristol University Earthquake Engineering Research Centre[3], undertaken on a vertically spanning wall subject to 1.0g lateral acceleration and results are shown in Table 1. It will be seen that significant differences exist.

Panel out-of-plane frequency may be established by several methods but generally standard flat plate formulae[5] were used. The general formula is dependent on edge boundary conditions, and values are shown in Table 2 for four common edge conditions.

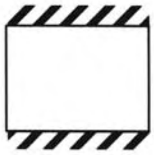
The Dynamic Amplification Factor (DAF)

To obtain a reliable estimate of the DAF, it is necessary to undertake rigorous calculations utilising standard methods[6]. Such methods should make full allowance for damping (the ability of a material to dissipate energy and stop vibrating), which can be taken as 5% of critical damping for brickwork masonry. The results of these calculations give a DAF of unity for static loading but some different value for dynamic transients. At the frequencies of the brick panels encountered, this factor was usually greater than unity, however, for high frequency loading it could be less than unity. The shape of the DAF v frequency plot will generally reflect the input transient shape (load v time) of the applied loading. A summary of maximum DAF's for standard cases is given in Table 3 for varying damping ratios. It can be seen that for the worst case loading encountered, applied loading could be amplified by a factor approaching four.

It is desirable to understand the variability of the DAF for a broad range of frequency values, especially when a hot gas release boundary may contain over 100 masonry panels. Therefore, for each blast transient, the DAF was evaluated over the possible frequency range of all panels to be assessed. A DAF v frequency plot may then be constructed, as depicted in Fig 4.

Fig 4. A DAF plot, derived from the pressure transient in Fig 3

Table 3: Variation of DAF with damping ratio for various transients				
Maximum DAF	**Transient type**			
Proportion of critical damping ξ	Triangular	Rectangular	Semi-circular	HOTJET
				
0	1.50	2.00	1.71	3.87
0.05	1.41	1.86	1.59	2.4
0.1	1.32	1.73	1.5	2.02
0.25	1.12	1.42	1.28	1.43

Table 4: Capacities of a two leaf, stretcher bonded, 3m × 5m panel ($\gamma_m = 3.5$ $\gamma_f = 1.4$)					
Dynamic blast load resistance P_{lat} (kN/m^2)					
102mm single leaf	Elastic bending	0.02	0.18	0.3	–
2 leafs + interleaf ties		0.05	0.36	0.60	–
2 leafs + composite ties		0.105	0.79	1.32	–
102mm single leaf	Vertical arching	–	–	–	0.97
2 leafs + interleaf ties		–	–	–	2.26

To account for the uncertainty of panel frequency calculation, the following procedure was adopted to select a DAF from the plot:

- Static DAF – computed using static material properties and appropriate to elastic conditions. The frequency was broadened by ±15% and the most onerous DAF selected from this range,
- Dynamic DAF – computed using dynamic material properties and appropriate to plastic conditions. The most onerous DAF was selected from a frequency range between the static and dynamic frequencies.

Methods of assessment

It was possible that existing brick panels were adequate to withstand hot gas blast loads. So, before undertaking any (undesirable) strengthening, two methods of lateral load capacity assessment were adopted. Both these methods are listed in BS 5628[7].

Elastic bending assessment

The elastic bending method is based on yield-line theory. Standard solutions for a range of edge conditions and aspect ratios (H / L) are available via a bending moment coefficient (α). The general equality to be satisfied is:

$$\mu \cdot \alpha \cdot \gamma_f \cdot (P_{lat} \cdot DAF) \cdot L^2 \leq \frac{f_{kx}}{\gamma_m} Z \qquad \text{...(2)}$$

where $\mu = f_{kx\parallel} / f_{kx\perp}$ is the orthogonal ratio (Poisson's ratio of masonry) and Z is the section modulus of the brickwork resisting the blast. The product, $P_{lat} \times DAF$ may be viewed as an equivalent static load.

Further assessments may be made using the same method and assuming new edge restraint conditions until the panel is deemed satisfactory.

This method generally predicted the panels to be unacceptable. The alternative approach, using arching, proved much more useful since it suggests significantly greater capacities.

Arching assessment

Arching in flat panels, as opposed to vaulted arches, assumes a 'virtual arch' is set up within the masonry thickness. The virtual arch thickness is taken as 10% of the masonry thickness (t). As deflection takes place, a three-pinned arch develops in the panel and hence a statically determinate failure mechanism exists. Although the panel is deflecting with the load, the virtual arch is springing against the load and a resistance mechanism is developed, this is shown in Fig 5. This is deemed to be a plastic approach and so the dynamic modulus is used in deflection calculation rather than the elastic modulus.

Arching is a compression system and is dependant upon resistance of the thrust generated by arch displacement. Consideration of edge conditions is therefore of paramount importance. BS 5628[7] currently refers only to horizontal arching but the principles are equally applicable to vertical arching.

Nuclear power station structures are generously sized, usually offering sufficient floor beam dead weight to resist thrust from vertical arching. However, where horizontal arching was used, the column at the end of a row of panels must provide sufficient stiffness to resist arching thrust. Significantly lower blast loads were predicted in the upper storeys of the building where resistance could be easily justified, however a rigorous capacity and deflection check was required in the lower storeys.

BS 5628[7] outlines this approach for panels of slenderness up to 25 (see

Equation 3). Where a greater slenderness was encountered, the arch deflection and rise (r) were calculated and a reduced capacity results (see Equation 4).

Two-way arching is not used but could potentially offer a considerable increase in panel capacity, if a code compliant method were available. It is believed that such a method would predict capacities similar to those observed in the British Ceramic Research Association test[2].

$$\frac{H}{t} \leq 25 \qquad (P_{lat} \cdot DAF) \cdot \gamma_f \leq \frac{108}{100} \cdot \frac{f_k}{\gamma_m} \cdot \left(\frac{t}{L}\right)^2 \qquad \text{...(3)}$$

$$\frac{H}{t} > 25 \quad (P_{lat} DAF) \cdot \gamma_f \leq 1.2 \frac{f_k}{\gamma_m} \cdot \frac{t \cdot r}{L^2} \qquad \text{...(4)}$$

Table 4 gives indicative dynamic overpressure capacities for brickwork panels of various boundary conditions and assessment methods. Clearly, arching methods predict significantly higher capacities than elastic bending methods but greater deflections occur.

The use of cavity grouting, to create a composite wall section was not pursued as previous work had highlighted the difficulty of successfully grouting whole panels without bursting the brickwork.

Remedial measures

When panels are deemed inadequate, many remedial solutions are available, some more utilitarian than others. However, since the majority of panels considered were within industrial plant, there was little consideration of aesthetics. The major consideration when selecting a strengthening solution was minimisation of disruption to plant and process.

In nuclear facilities, a HAZOP (HAZard And OPerability study) is routinely undertaken in conjunction with the plant operation team, to ensure that the implications of any proposed modifications are fully understood and to ensure any residual risks involved are ALARP (As Low As Reasonably Practicable).

Solution optioneering was used extensively on this project. This involved identifying every possible solution for every panel, the team could then identify the safest solution to implement. The selection would typically centre upon considerations of possible live plant damage (nuclear safety), access availability and difficulty, radiological exposure and knock-on effects for plant operation. Once this process was completed a written Safety Case was submitted for rigorous Independent Nuclear Safety Assessment, upon approval implementation could begin.

The design process for panels requiring modification is cyclical. Firstly, the assessment of capacity is dependant upon the boundary conditions to establish panel frequency and hence the DAF. But thereafter should the panel fail its capacity assessment, any modification will change the boundary conditions and hence frequency and hence the dynamic blast load that must be resisted. Thus the design process is iterative.

It is important to observe the relationship between structure stiffness and response frequency for each blast transient. A panel may be rendered acceptable simply by increasing its stiffness (without altering its mass) so significantly increasing its frequency and hence reducing its DAF. Conversely, since frequency is a function of (stiffness / mass), adding mass may have little benefit. It may even worsen response.

Arching

Creating conditions to permit arching offers the solution with potentially the least intervention since providing solid packing between the panel masonry and the surrounding structure may be the only modification required.

Several simple measures may be taken when arching provides an acceptable assessment:

- When vertical arching is acceptable, the gap between the top of the masonry and the building frame was dry-packed with non-shrink sand : cement grout. This ensures a load path for arch thrust. Where the gap was less than 2mm, no modification was required as the panel locks itself in as it bodily rotates under blast load[3],
- When horizontal arching is acceptable, vertical edge restraints, usually angles, were fitted to the building frame by bolting or bonding to the concrete frame. This prevents bodily movement of the panel until the arch is developed.

These solutions are materially cheap but often access and labour intensive. Figs 6a & 6b and Fig 7a & 7b respectively depict modifications undertaken for vertical and horizontal arching.

Panel sub-division

When arching was not an acceptable solution, usually due to the presence of significant panel openings or service penetrations, a fully elastic solution was required to provide the panel with sufficient resistance.

Introducing vertical posts to sub divide the masonry panel was often chosen. Choosing a post spacing which forced a sub-panel aspect ratio (H/L) greater than 1.75 increased the strength of the panel by a factor of three since the masonry was then forced to span horizontally rather than vertically.

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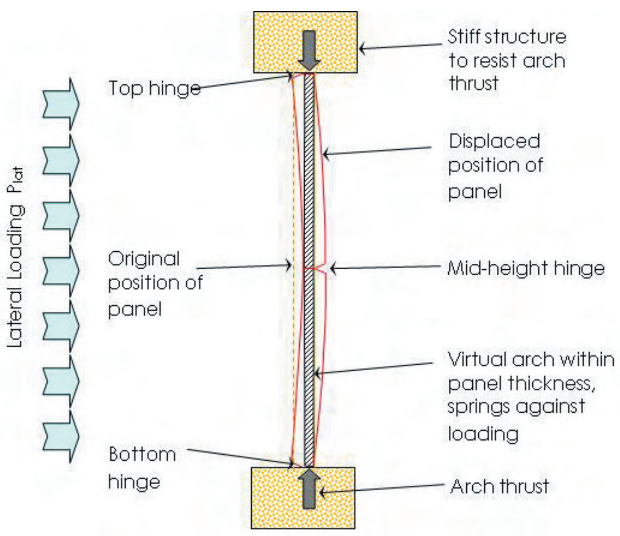

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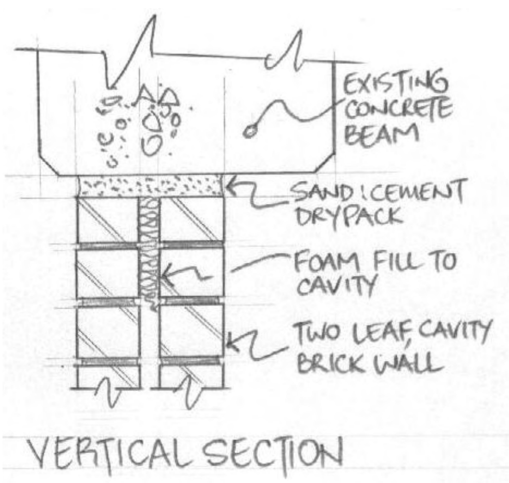

Fig 5. Assumptions for the 'virtual' arch in flat panels / Figs 6a,b. A dry packed top edge restraint for vertical arching

7a

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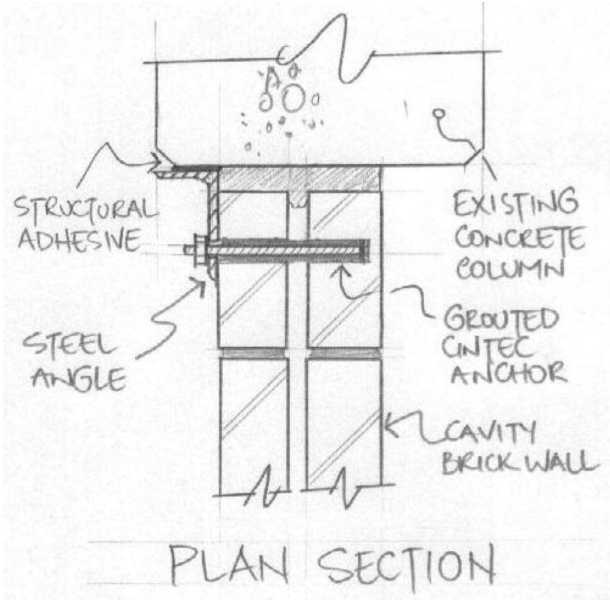

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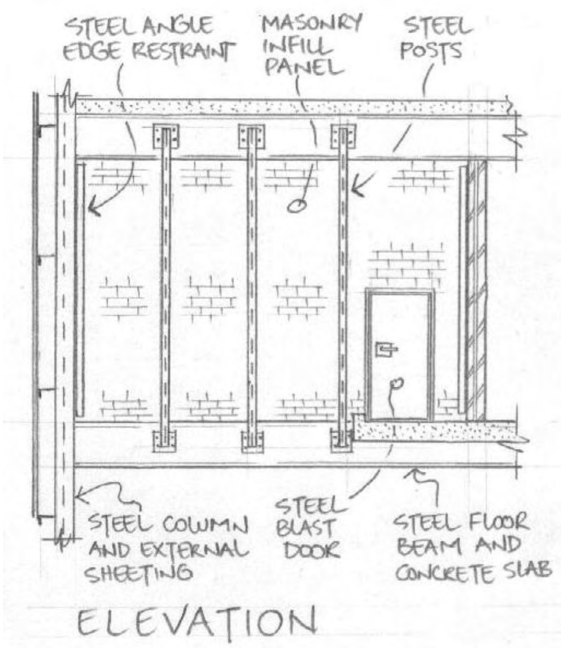

9a

9b

Figs 7a&b. A steel angle bonded to a concrete column and (CINTEC) anchored to the edge of a masonry panel to form a vertical edge support for horizontal arching / Figs 8a&b. A wall strengthened by dividing into horizontally spanning panels, adding interleaf ties, edge restraints and a steel blast door / Figs 9a&b. A post head connection fixed using structural adhesive, and base connection let into the floor screed to avoid trip hazards

Vertical steel posts were fixed top and bottom with base and head plates respectively, into the surrounding structure. These posts were generally universal beam or column sections. To ensure the strengthening responds under blast load, it must be stiffer than the masonry panel being supported and this was achieved using tight deflection limits (span / 500) for post selection.

Dependant upon the direction of loading and post position, ties were needed between the masonry and posts to transfer blast loading out of the masonry and to complete the load path. Using posts was an expensive and unsightly solution, but often the only effective one.

Figs 8a & 8b and Fig 9a & 9b depict a panel strengthened by sub-division and the connection details used at the top and bottom of the post.

Cavity wall ties

Since the majority of wall panels in this project were of two separate stretcher bonded leaves, an obvious consideration was to introduce cross-cavity ties as strengtheners. Remedial ties may be used to force multiple leaves to share load, so resisting the blast load more effectively. Ties were often introduced in conjunction with the other modifications discussed earlier in this text. They may be installed from either one, or both sides of a wall to avoid wall mounted plant. Off the shelf products such as Helifix screw-in ties were used at very modest cost.

Where interleaf ties did not provide an adequate solution, a more substantial tie was used. The object was to bind the separate leaves more effectively so as to develop composite action rather than just dispersing load between the leaves. A proprietary masonry tie manufactured by CINTEC was found to be effective. Site testing was used to establish wall capacity since the composite action was dependant upon limitation of slip between the separate brick skins. Using 12mm diameter CINTEC anchors, grouted into 30mm core drilled holes, achieved around 80 % composite action. Although these anchors provide a significant increase in wall strength they are also considerably more expensive than simple interleaf ties. Fig 10a &10b shows an installed CINTEC tie.

The options for assessment of masonry panels, and subsequent strengthening are summarised in Fig 11.

Implementation works

The importance of safety in a nuclear installation cannot be underestimated. Although conventional site health and safety will not be discussed here, there are a number of nuclear related safety issues which are worth mentioning.

Restricted access – often requiring multistorey access systems to be

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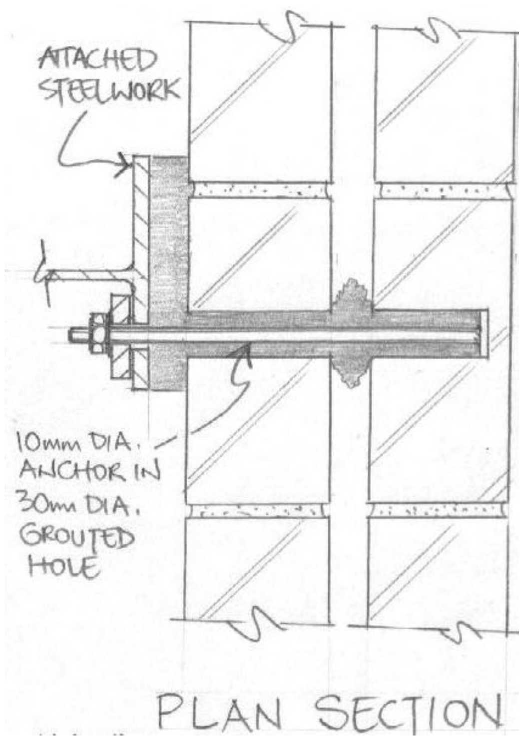
10b



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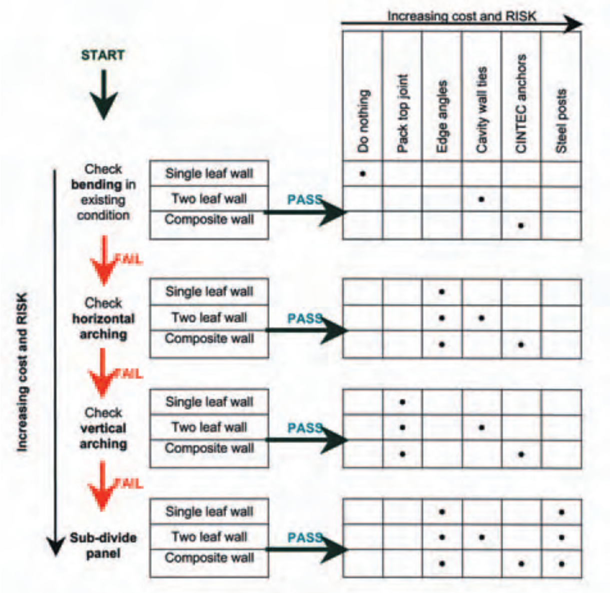

Figs 10a,b. A 10mm diameter CINTEC anchor, following grout injection / Fig 11. Flowchart depicting panel assessment and strengthening options

threaded between safety critical plant, which makes this a slow and meticulous process. An example of this is shown in Fig 12.

Safe systems of work – designed plant protection was often needed since damage to on-line plant could have created the hot gas release that the work was intended to prevent. This is shown in Fig 13, which shows the two stage protection to a safety release valve in order to strengthen the adjacent masonry wall with steel plates. Notwithstanding this, when considering the question of a hot gas release during implementation works it is rather sobering to consider that the short time at risk is an adequate justification for the risk to personnel.

The need to avoid damage to existing concrete reinforcing bars often meant that the use of anchor bolts was impossible. Many connections between strengthening and the building frame were therefore made using structural adhesive.

To complete the strengthening of a masonry panel, it was often necessary to seal penetrations where services passed through. In many cases this involved complex details of steel plating, Fig 14 shows such a case. To minimise the effect of service penetrations in masonry, a proprietary fire proof sealant was specified whose characteristic compressive strength exceeded that of class I mortar.

Nuclear power stations are well engineered, so restrictions on working adjacent to hot or noisy plant had already been dealt with by plant operators.

Radioactivity exposure is inevitable in a nuclear installation but use of environmental and personal monitoring combined with limited time working ensured that stringent exposure levels were never breached. No work was undertaken in contamination controlled areas.

Conclusion

Dealing with blast loading on masonry panels is a formidable technical challenge requiring knowledge of blast transients and the loading pressure time pulse plus a knowledge of effective pressure as determined by a panel's dynamic characteristics. Since these characteristics are sensitive to design assumptions, a bounded approached to design is required. Techniques are available to calculate the applied dynamic loading in terms of an amplified static response.

These techniques require knowledge of the natural frequency of the masonry panels. This frequency is difficult to establish with confidence, hence due allowance for the variability of actual edge conditions and construction must be made.

The available experimental evidence suggests that masonry panels can resist seemingly very high blast loads and this is explainable by the impulsive nature of the loading. To assess panel strength theoretically is more difficult but the arching method given in BS 5628 is a useful technique. It also highlights that in remedial work, resistance is best achieved by providing the boundary conditions that will facilitate arching action being realised. Moreover, where arching can be substantiated, this will generally lead to the most economical solution for panel 'strengthening'. It does however require a substantial surrounding structure to deal with the reaction thrust.

When arching action cannot be justified, panel flexural strength can be increased by bonding leaves together using externally inserted anchors. Test evidence shows there is a limit to the degree of composite action that can be developed and this is governed by the amount of slip that occurs on the anchor brick interface and the span of the panel.

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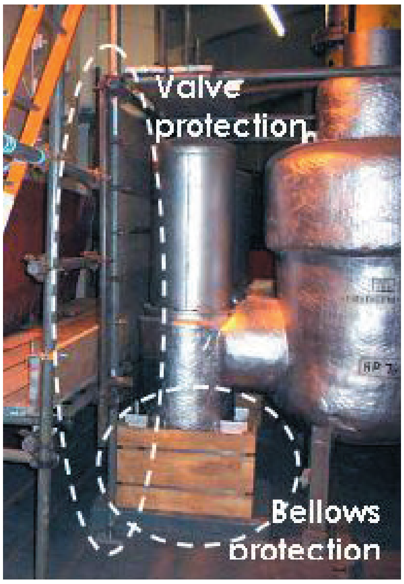

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Fig 12. Plant congestion and access problems over three storeys of masonry panel / Fig 13. Wall plating to prevent bed-joint scour at a safety release valve / Fig 14. Steel platework to seal penetrations in a masonry panel