

# DESIGN OF BLAST RESISTANT BUILDINGS IN AN LNG PROCESSING PLANT

Troy Oliver<sup>1</sup>, Mark Rea<sup>2</sup>

## ABSTRACT:

*This paper provides an overview of the work undertaken in the design of multiple buildings for one of Australia's largest liquefied natural gas processing plants. The design brief required that all buildings located on the site were to be designed for blast resistance, due to the potential for a vapour gas cloud explosion to occur in the facility. The site location dictated that the buildings were subject to some of the highest cyclonic wind loads considered in Australia. The paper discusses the concept of blast design, describes some of the design involved with the plant, compares the relative effect of blast and wind loading on the buildings, and provides comment on the differences in structure types under these loading conditions.*

**KEYWORDS:** Blast resistant, LNG plant, finite element analysis, plastic design, post yield.

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<sup>1</sup> Troy Oliver, Pritchard Francis Civil and Structural Engineering Consultants. Email: troy.o@pfeng.com.au

<sup>2</sup> Mark Rea, Pritchard Francis Civil and Structural Engineering Consultants. Email: mark.r@pfeng.com.au

# 1 INTRODUCTION

With a rapidly expanding Liquefied Natural Gas (LNG) industry in Western Australia, safety and risk management processes are seeing an increased number of buildings having a requirement for “blast resistance”. LNG plants store large volumes of hydrocarbons and other fuels which are highly explosive and pose a risk to human life. In 2010, Pritchard Francis undertook a new project (The Project) which entailed the design and documentation of multiple large buildings in one of Australia’s largest LNG processing plants. The proximity of the building sites to the processing facility meant the buildings were required to achieve certain levels of blast resistance in order to satisfy the Client’s requirements.

Located in the North-West region of Western Australia, the design was subject to significant cyclonic wind loading, along with the standard requirements for earthquake and gravity loading as per the relevant Australian Standards. Preliminary schematics were developed based on simplified design techniques, with the eventual designs being modelled as entire buildings under the dynamic blast loading. Investigations were undertaken during the early stages of the project in order to establish the most efficient type of building structure.

## 2 BLAST DESIGN

### 2.1 DESIGN PHILOSOPHY

The underlying philosophy of blast resistant design is to ensure that persons who are within a building are not subjected to a higher level of hazard than those who are not [1]. Past examples, including two major explosion events in Texas, USA in 1989 and 2005, provided evidence of fatalities being caused by collapse of buildings in a blast event [1]. Other key factors in assessing the requirement for blast resistance of a building are both the required function of the building in a blast event, and the financial implications a blast would have with respect to the building’s contents and purpose. A building required for emergency services, or to control plant operations after a blast will have different requirements for blast resistance to that of a warehouse type building; however, it is still a requirement for either type of structure to avoid collapse and subsequently endanger the lives of its occupants.

While a structure is typically designed based on prescribed blast loading conditions, calculating a blast load in an explosion is subject to many variables and difficult to accurately define. To allow for the uncertain loading there are several factors which must be considered quite differently

to a standard design approach. The principles are centred on providing redundancy within the structure by designing for ductile failure mechanisms. This produces a requirement for the shear resistance of an element to be greater than its flexural resistance, ensuring the member fails in the more ductile failure method of bending [1]. A second consequence of this requirement is that connections are typically designed for greater than the member’s capacity. In some cases where the connection or shear capacity were not sufficient to satisfy this, a solution was achieved by reducing for example the gauge of a beam to reduce flexural capacity, while maintaining the depth to retain shear resistance.

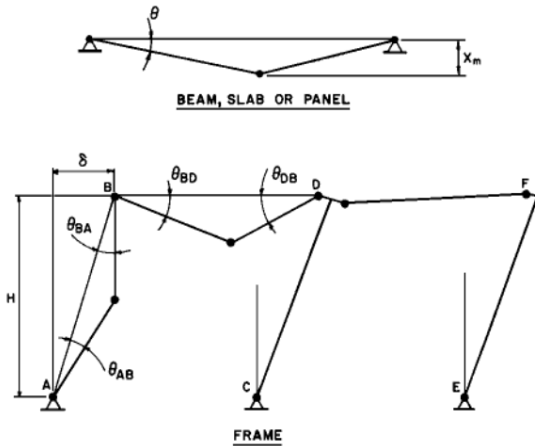
A key factor in blast design is the need for energy absorption. In order to allow a building to remain standing after a high energy blast wave, plastic deformations are permitted, and the performance requirements are based on the building usage and importance level. The performance requirements are defined by a Response Range. The three response ranges are defined as low, medium and high, with low being the most stringent criteria. Each response range corresponds to an expected level of damage in Table 1 below [1].

**Table 1: Building damage levels**

Response Range	Description of expected damage
Low	Localized component damage. Building can be used; however, repairs are required to restore integrity of structural envelope. Total cost of repairs is moderate.
Medium	Widespread component damage. Building should not be occupied until repaired. Total cost of repairs is significant.
High	Key components may have lost structural integrity and building collapse due to environmental conditions may occur. Building should not be occupied. Total cost of repairs approaches replacement cost of building.

The two parameters used to measure blast performance are the allowable ductility ratio and hinge rotations. Ductility ratio is a measure of the amount of post yield deformation experienced by the member, and is calculated as the maximum deflection of the member divided by the maximum elastic deflection [1]. This is a parameter which can

be measured for simple elements; however, in the case of overall building frames, it is hard to define the maximum elastic deflection. Consequently, a limit on the side sway is specified for frames as a fraction of the building height, similar to a typical building sway limits for serviceability limit states. Hinge rotations are simply measured as the angle between two lines drawn from the hinge to the point of maximum deflection and the opposite member end as depicted in Figure 1 [2].



**Figure 1:** Member end rotations for Beams and Frames [2]

## 2.2 ACCIDENTAL LIMIT STATE

Whilst traditional design is commonly carried out as an Ultimate Limit State (ULS) case, blast design is considered an Accidental Limit State (ALS). In this case, a different approach is taken with respect to the loads and material properties.

In most design resources, material properties are stated as minimum requirements. The average mechanical properties of materials will generally exceed these. To avoid conservatism and obtain a more realistic representation of a materials response, a Strength Increase Factor (SIF) is employed to account for this difference [1]. This is in contrast to ULS design where a material reduction factor is used to reduce the strength of materials. In addition to the SIF, material factors are generally increased additionally by a Dynamic Increase Factor (DIF) to account for an increase in strength under dynamically applied loads. Tables are readily available which specify appropriate SIF and DIF values for various materials under certain actions (flexure, tension, shear etc.) [1].

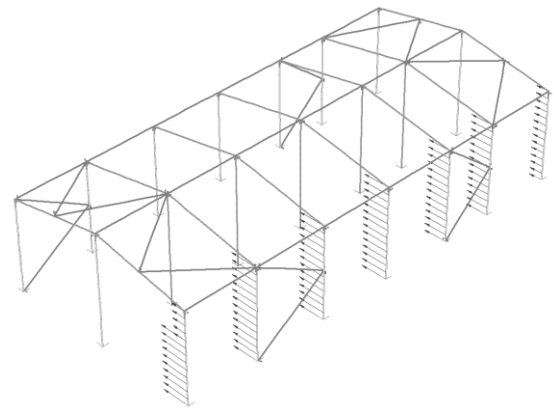
## 2.3 METHODS OF ANALYSIS

As a blast load is dynamic, the response of a particular element is dependent on its stiffness and mass. Consequently, even simple elements must be analysed dynamically, most commonly using the single degree of freedom (SDOF) system. This

method idealises an element as a lumped mass ( $M$ ), with a given spring stiffness ( $K$ ) and damping constant ( $C$ ), and an equivalent load is determined based on the elements loading and boundary conditions. The SDOF system is expressed below in Equation (1) where  $F(t)$  is the blast force as a function of time.

$$F(t) = Ma + Cv + Ky \quad (1)$$

One effect that is not accounted for in this technique is the interaction between elements. This is deemed to be negligible when the natural frequencies of connected members differ by a factor of two or more [3]; however, when this is not the case, members are required to be analysed as a Multiple Degree Of Freedom (MDOF) system. Whilst this can be achieved by numerical methods, the most common approach is to analyse a structure using a finite element analysis (FEA). This allows for an entire structure to be modelled and a load path can be more accurately followed than when a series of SDOF assumptions are made. An example model is depicted in Figure 2.



**Figure 2:** MDOF steel building frame FEA model

Providing elements are not to be treated as coupled, loads can be applied to a structure based on typical tributary width assumptions. This approach was taken to the design of elements such as purlins, roof sheeting and some rafters which did not form part of a portal system. Light gauge steel purlins were assessed based on the SDOF method, with 300 deep Z-purlins being used for the majority of the buildings, spanning between 8-10m in most cases.

## 3 THE PROJECT

### 3.1 SCOPE OF WORK

The scope of the Project included the design and documentation of multiple large, non-process buildings in various locations around the LNG

plant site. The buildings varied in size and construction due to mixed usages and the proximity to the processing facility which is the potential source of blast. Of the multiple buildings designed, three buildings which represent the different types of construction used are discussed further. The largest building (Building A) is an operations centre, housing high staff numbers and large amounts of critical equipment, and hence required designing to the most stringent blast category. Building B is a large maintenance shop and Building C was a smaller operations building within the processing area.

The design brief specified that buildings were to be designed for blast loading along each building's major and minor principle axes, adding a further degree of complexity to the design. Along with the requirement for blast resistance, the buildings were also subject to high cyclonic wind loads, due to the site being classified as Region D in accordance with AS1170.2: Wind Actions.

## 3.2 DESIGN PARAMETERS

### 3.2.1 Blast Criteria

Whilst the Project encompassed several large buildings, the focus of this section is centred on three buildings, A, B and C. Each of these buildings was subject to different design criteria as defined by the Client (Table 2). As a result of the varied loads, building geometry and functions, these three buildings were designed differently, albeit based on the same underlying principles.

**Table 2:** Blast parameters

Building	Blast Overpressure	Blast Duration	Response Range
A	15 kPa	150 ms	Low
B	15 kPa	150 ms	Medium
C	100 kPa	22 ms	Low

### 3.2.2 Building A

Building A is the primary building in the scope of the Project. With plan dimensions of over 50m by 100m, the building is a large centre housing significant amount of control equipment, essential to plant operations, along with being extensively staffed during standard plant operations. As a result of its importance, and requirement to remain operational after a blast, it has been designated as low response range. The building was to be fully sealed and cyclone rated to achieve a high level of safety in any event.

The building comprises two roofs, with a lower concrete roof slab forming the blast envelope, and an upper roof providing a typical waterproof layer. The upper roof is of typical construction

comprising steel sheeting fixed to light gauge purlins and supported off hot rolled rafters. The upper roof is supported directly off the lower slab. The lower roof slab is poured on metal decking and is supported off composite secondary beams which span between steel portal frames acting along one principle axis of the building. The steel frames are designed to eliminate the need for temporary propping in order to assist in construction. In the perpendicular direction, the roof diaphragm spans across the shorter plan dimension of the building back to concrete shear walls. The walls are constructed in precast concrete panels, which are tied together structurally to transmit the lateral shear force along two sides of the building. The columns are founded on pad footings, with strip footings spanning between pads to provide continuity and also engage additional passive and frictional resistance to horizontal loads.

### 3.2.3 Building B

While designed to the same blast overpressure as Building A, the design brief for this building was substantially different. Serving as a maintenance shop, this building has a large open working area. With large roller doors open for vehicle access in normal operations, it was assumed that the doors would be open in a blast event, exposing the building to internal blast pressures.

The building was designed as a typical "shed" type structure, with large steel portal frames supporting light gauge steel purlins and roof sheeting, and steel bracing being provided in the direction perpendicular to portal frames. The wall cladding comprised precast concrete panels. Due to the presence of large roller doors which did not comply with the impact rating requirements of AS1170.2, the building was designed for full internal wind pressure in cyclonic conditions. This led to very large sled footings along the portal frames, with footings being linked by ground beams to help engage greater frictional and passive resistance.

### 3.2.4 Building C

Due to the proximity of this building to the processing areas, the previously adopted design approach of concrete panels and steel frames supported by pad footings was not feasible. As this building was comparatively small in plan dimensions (approximately 10m by 15m) it was able to be designed as an insitu concrete box. The walls span to a concrete roof diaphragm which is supported by concrete insitu shear walls. Due to the high lateral loads (over 270 kPa reflected blast pressure), it was necessary to support the building on a raft slab, with steel driven piles being designed as lateral supports.

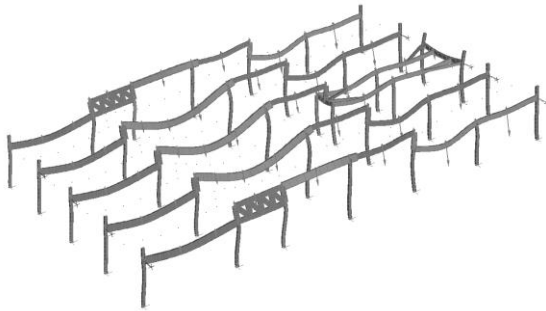
As the steel piles are not infinitely stiff and are subject to some deflection along the length of the

pile it was necessary to model the entire building, including raft and piles, in a single model. While requiring large computational resources, this was deemed to be the most appropriate design method due to the interaction of the foundation response and building response.

### 3.3 DESIGN APPROACHES

#### 3.3.1 Building Frames

As discussed previously, when several elements are supported such that their responses are coupled, the entire frame is modelled in order to reduce inaccuracies. For all buildings, a large building model was created used the FEA software Strand7. The model was analysed dynamically, and each members performance assessed individually with respect to the limits imposed in ASCE (2010). An example frame is shown in Figure 3, which is a section of Building A modelled in the Strand7 analysis package.



**Figure 3:** Building A finite element model

Buildings consisting of a series of repeated portals were able to be represented by a single portal model in the early design stages; however, all buildings were eventually modelled as global framing models to capture the behaviour of the structure along both major principle axes. Some buildings requiring clear spans of 24m required portals comprising 900WB257 rafters with 800WB146 columns.

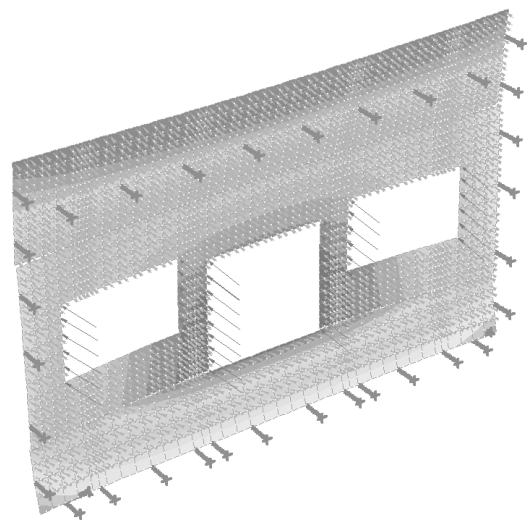
In the case of Building C, the entire building was modelled together due to the connections and compatibility between elements. As the building was a stiff concrete structure in order to withstand the significant blast loading, the foundations (piled) were required to be modelled as they interacted with the raft slab foundation.

#### 3.3.2 Concrete Panels

As the natural frequencies of the precast concrete panel cladding and the typical steel frames differed by a factor of more than two, they were able to be analysed separately. In order to accurately represent the internal force distribution in the panel however, edges supported by columns were modelled with springs in contrast to pin supports representing the

more stiff support provided by direct fixings to strip footings. Concrete panels were also modelled in the Strand7 FEA package.

One difficulty encountered in modelling the wall panels was in representing the composite nature of the reinforced concrete, while also considering the difference in blast capacity of the reinforcing steel and concrete. The panels were modelled as homogenous material 2D plate elements in Strand7. The panel stiffness was taken as the average of the gross and cracked section properties in line with ASCE (2010). A plate thickness was chosen to give a rectangular section with the required stiffness. Material yield strength was then set based on a plastic cross section achieving the calculated blast capacity of the panel (including SIFs and DIFs).



**Figure 4:** Precast panel FEA model

Typical spans and panel thicknesses were modelled for each building, with more detailed analysis undertaken for panels with irregularities such as doors, windows and mechanical penetrations. A typical wall configuration with windows and a door is shown in Figure 4. The most common panel thickness is a 250mm thick panel, spanning up to 8-10m on several buildings. Large panel thicknesses were required to achieve adequate blast capacity in walls which required penetrations for mechanical equipment and other services.

A dynamic blast loading analysis was carried out for each panel, with the panel being checked for allowable deflection criteria. Having satisfied the blast criteria, the shear capacity was checked in accordance with standard design procedures as per AS3600 (while omitting safety factors and including SIF and DIF multipliers) to ensure a ductile failure would result if the panel were loaded beyond the design pressure.

### 3.3.3 Foundations

With the exception of piles for Building C (as described in section 3.2.4), foundations were typically analysed based on peak dynamic reactions taken from the FEA frame models. Designing statically for peak dynamic reactions is a conservative method; however, footings were one element which was often governed by wind actions due to the presence of uplift forces which are not imposed by blast actions.

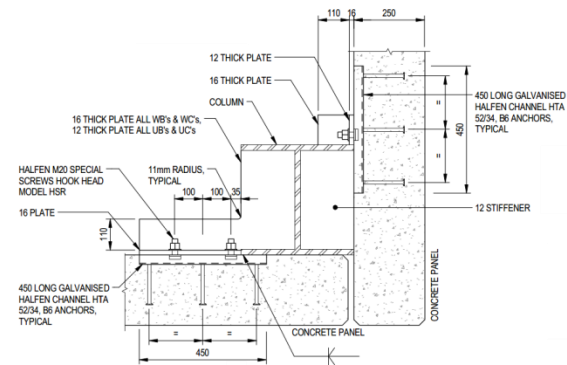
Due to the ground conditions in some parts of the site, some buildings were required to be founded on driven steel piles, though most areas were suitable for spread pad foundations. Strip footings were required to the perimeter of each building for several reasons, primarily to act as a foundation for the precast panels, but secondly to join pad footings between grid lines and allow bracing loads to be spread over multiple pad footings.

## 3.4 DETAILING

### 3.4.1 Lateral Restraint and Movement

One of the key requirements for blast design, based on the ductility requirement, is to ensure that members are able to develop their full plastic capacity. In standard steel designs, member capacities are commonly governed by buckling, and are not able to utilise the full moment capacity of a section. Under blast loading, members are required to be fully restrained in order to form plastic hinges and behave in a ductile fashion. For rafters, this is achieved through the use of fly braces, a standard detail which connects the unrestrained flange of a rafter to a purlin in order to prevent lateral movement of the compression flange. This detail has been used on all purlins which have a blast rating requirement, with fly braces spaced at lengths not greater than that required to achieve section capacity.

In a typical steel clad building with wall girts and sheeting, a similar detail can be applied to the columns; however, buildings A and B are constructed of steel portal frames clad with precast concrete panels. The steel portal frames are designed to absorb blast energy through large deflections which cause the formation of plastic hinges, whereas the concrete panels are stiff elements not able to accommodate large movements. To maintain compatibility between these two elements, a connection was detailed which would allow relative movement between columns and panels, but prevent twisting of the column flanges as shown in Figure 5.

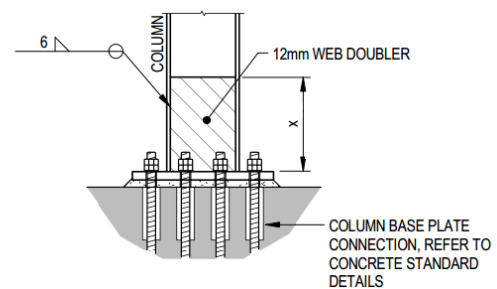


**Figure 5:** Corner column restraint detail

The detail is arranged with proprietary sliding channels cast into the precast panel, which are then bolted to the column cleats once panels are installed on site. This allows the column to sway laterally in a blast and undergo large deflections in the portal direction, but to remain laterally restrained at both flanges.

### 3.4.2 Connections

The requirement in blast to have greater shear capacity than the maximum flexural resistance of the member presented several design challenges. One such issue was apparent in several UC columns supporting external concrete panels. Many of the columns are 250UC and 310UC columns which are not spanning significant heights. As a result of the shorter spans, the resistance of the member is high in flexure, and this resistance load must be resisted in shear at the ends. This was achieved by adding a web doubler plate (Figure 6) to the base of the columns, which provides a greater thickness of web to increase the shear capacity at the supports.



**Figure 6:** Column base web doubler detail

### 3.4.3 Construction Requirements

As part of the design and construct team delivering the structural engineering input for the project, a heavy focus was placed on construction requirements from early in the design stage. One key issue was the need for temporary stability with a minimum amount of additional support. Being the most critical building on the job, Building A was

heavily influenced by timing and construction factors. The concrete roof slab was designed to span between supports without the need for propping, while the general construction methodology was designed to allow precast panels to be installed with a minimum amount of temporary bracing.

## **4 BLAST AND WIND LOADING COMPARISON**

### **4.1 LOADING AND GEOMETRY**

When designing to AS1170.2, designers will often refer to “basic wind pressure” and have a good grasp on what pressures are considered large and small. As blast loading is a less widely understood and documented field of structural design, it is important to understand how blast may compare to wind pressure when considering governing load cases. The majority of buildings in the Project were designed for a 15 kPa blast overpressure of 150 ms duration. The following comparisons are provided in order to demonstrate the maximum wind load that a typical portal and braced frame could withstand when designed for the 15kPa blast for the low allowable response range.

The building frame being used for comparison is of a typical shed type construction, similar to that of Building B. The steel structure comprises a series of 530UB92 portal frames, with central 310UC97 columns supporting the rafters at midspan. The building is 14m wide in the portal direction by 30m long in the braced direction and 7m high, with a tributary width of 5.2m. The roof pitch is 10°. An isometric view of the frame is shown in Figure 2. The bracing system consists of 219.1 x 4.8 CHS roof and wall bracing members. A smaller 168.3 x 4.8 CHS joins the frame between the braced bays at each end of the building.

### **4.2 RESULTS**

The portal frame sizes are governed by the building sway criteria under blast loading; however, when considered under wind loading conditions, the portal frame size is governed by the maximum bending moment developed at the base of the frame. Section capacity of a 530UB92 is 640 kNm. Based on this value, the frame would be adequate for a basic wind pressure of 14.9 kPa.

When providing the comparison for a braced frame, the equivalent wind load has been considered with respect to the bracing, as the comparison seeks to compare the lateral load resisting capacities in wind and blast.

As compression bracing is typically governed by buckling rather than yielding of the steel in compression, the design of bracing elements is

done based on peak dynamic member actions. The bracing element size is therefore based on selecting a member with greater blast capacity than the peak dynamic axial force. As a result, there is inherently a factor of safety in the blast capacity, and this factor of safety was maintained for the wind case in order to provide more accurate comparisons.

The bracing sizes were governed by the wall bracing struts, which are a 6.3m long 219 x 4.8 CHS. The factor of safety in blast for this member is approximately 1.4, with a peak dynamic axial force of 548 kN and a blast capacity of 749 kN. To achieve this same factor of safety in wind, the maximum basic wind pressure that could be applied to the building is 32.8 kPa.

As can be seen from the values produced from this comparison, the braced structure exhibits double the strength of the portal structure when subjected to a static design load. This is due to the higher stiffness of a braced structure which attracts a larger amount of load than a flexible portal frame.

## **5 CONCLUSIONS**

Due to the high profile nature of the Project, and the extreme loading conditions associated with the buildings, the designs were subjected to a large amount of scrutiny, with internal design checks, third party design reviews and also a thorough design review by the Client. Whilst the blast loading case governed the majority of the structural elements, many aspects were still influenced by other design actions and were also assessed individually.

Whilst the loads imposed due to blast were significant, there are many requirements which are added to the design and documentation of a building when blast resistance is required. Particular attention is required in detailing as there are many requirements for ductility and connection capacity which are not always standard practice. As highlighted by the comparisons made between wind and blast loading in this paper, it is crucial to ensure that blast resistant structures are designed and detailed with a focus on ductility and taking advantage of post yield behaviour, which is efficient in absorbing energy from a blast.

## **ACKNOWLEDGEMENT**

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## **REFERENCES**

- [1] The ASCE Task Committee on Blast-Resistant Design: Design of blast-resistant buildings in petrochemical facilities (second edition). American Society of Civil Engineers, Virginia, USA, 2010.
- [2] Department of the Army, Navy and Air Force: Structures to resist the effects of accidental explosions UFC 3-340-02. Unified Facilities Criteria, 2008
- [3] Biggs J. M.: Introduction to Structural Dynamics. McGraw-Hill, 1964