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INNOVATIVE DESIGN OF STRUCTURAL CLADDING SYSTEM

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Abstract

Following the Tohuku earthquake in Japan on the 11 of March 2011 a review of the UK's nuclear industry has been carried out using the lessons learnt from the Tokyo Electric Power Company (TEPCO) Fukushima-Daiichi station. The review concluded that there were no fundamental safety weaknesses in the UK's nuclear industry but nevertheless, the business agreed to improve for safety critical buildings to be resilient to tornado wind generated missiles, explosion and seismic loads.

In the European market, buildings that are required to perform critical safety functions have been traditionally constructed using thick reinforced concrete wall sections resulting in heavyweight structures. In addition a large footprint, labour intensive and long-time construction results in a high cost arrangement per meter square. To satisfy the requirement a new structural cladding system has been developed to absorb the high energy associated with impact, explosion and seismic loads using non-linear design techniques that consider post yield behaviour.

The structural cladding system is added to the conventional architectural cladding system. The structural cladding system has been developed using a steel plate spanning between UC sections cladding rails supported by the mainframe steelwork columns. The members forming part of the structure are subjected to different beyond design basis events such as explosion, tornado pressure, tornado generated missiles and seismic loads. In order to design the structure different approaches have been followed dependent on the load case under consideration. For example the blast and impact design capacities are governed by the rotation limits while the seismic design capacity is governed by ductility limits. To obtain an efficient design an iterative process has been carried out using several push-over and time-history analyses to reach to the optimal balance between the two above requirements.

The non-linear approach has enabled the design of a highly resilient building using an efficient steelwork braced frame superstructure fulfilling spatial, cost and construction requirements. The innovative and efficient solution developed can be tailored to suit alternative requirements allowing for a broad range of application where the absorption of energy by the ductile behaviour of the cladding system is required, such as in the oil and gas sector where there is potential for extreme blast combined with seismic loads.

Keywords: Safety critical buildings, beyond design basis events, structural cladding system.

1. Introduction

Following the Tohuku earthquake in Japan on 11th March 2011, a review of the UK's nuclear industry has been carried out by the Office for Nuclear Regulation (ONR) using the lessons learnt from the Tokyo Electric Power Company (TEPCO) Fukushima-Daiichi station. This review stated that there were no fundamental safety weaknesses in the UK's nuclear industry, but also concluded that using the lessons learnt from the event the industry can be made even safer. A document titled 'Japanese earthquake and tsunami: Implementing the lessons for the UK's nuclear industry' (ONR, 2012) was produced after the ONR review. This document identifies the requirement for an emergency equipment store.

The main functional requirement of an emergency equipment store is to provide a facility to store and maintain back-up and emergency response equipment that may be required by emergency responders for the nuclear power station. Vehicles and equipment may be required to respond to Beyond Design Basis (BDB) events, Design Basis events, and to non-nuclear related events on site. In particular, the facility has to be able to perform its main functions following a BDB event, and the equipment stored within the facility must be protected so that it is able to function following the same event. The design life of the building is 70 years.

Buildings that are required to resist Beyond Design Basis (BDB) events have traditionally been constructed using thick reinforced concrete wall sections. As a consequence they are heavyweight structures with a relatively large footprint, are labour intensive to construct and have relatively long construction times. These considerations result in a relatively high cost per m^2.

In replacement of the thick concrete structure, a structural cladding system has been developed using non-linear design techniques considering post yield behaviour to absorb the high energy associated with impact and blast loads. This design methodology enables to provide a highly resilient building using an efficient steelwork braced frame superstructure.

This paper describes the structural form of the emergency store together with the applied loading and methodology approach developed to design the steel superstructure and structural cladding system of the emergency centre. Due to the restrictions of the case of study presented only the methodology approached is described and no dimensions or results are provided from the analysis.

2. Structural Form

The emergency store building is a single storey steel framed structure. The structure has two cladding systems: a conventional architectural cladding system and a structural cladding system consisting of hot rolled UC sections and plain steel plate. The superstructure is supported on a thick reinforced concrete raft as shown in Fig. 1.

The steel frame is braced in both the north-south and east-west directions to provide lateral stability. Plan bracing in the roof transfers loads into the vertical braced bays. Chevron style bracing is used for both the vertical and plan bracing. There are four bays of vertical bracing in both the north-south and the east-west directions.

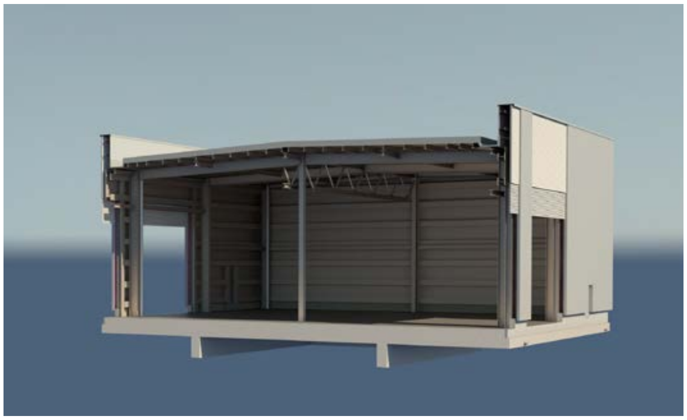


Fig. 1 – Cross section from BIM model showing building construction

3. Design Loads

The required performance of the emergency store is to remain functional following a BDB event. It is therefore necessary to establish and quantify the applicable BDB events to ensure the building design demonstrates resilience after a BDB event. It should be noted that the BDB requirement applies for natural external hazards, but not man-made hazards such as explosion.

The building is required to remain fully functional after the following loads (the magnitude and annual probability of the loads are based on a recent case study of the UK nuclear industry):

- Dead Loads (permanent action)
- Imposed Loads (variable actions)
- Snow Loads, annual probability of 1 in 50 years and 1 in 10,000 years
- Wind Loads, annual probability of 1 in 50 years and 1 in 10,000 years
- Tornado Loads, a T5 for the design basis event and T7 tornado for the beyond design basis event. The loads resulting from these tornados are outlined below:
  - Velocity pressure
  - Differential pressure
  - Tornado blown missiles
- External explosion loads, the external explosion load has been defined as an incident pressure wave in the free field arriving from any direction (one direction at a time). The magnitude of the pressure load has been taken from real cases of the nuclear industry requirements. The external explosion load is illustrated in Fig. 2 below. The pressure wave generated by the explosion would tend to propagate radially from the source and so would not hit the plane face of the structural cladding simultaneously along the full length of the building elevation. No position has been given for the source of the explosion, however; therefore it has been assumed, conservatively, that the explosion pressure load is applied to all elements on an elevation simultaneously.

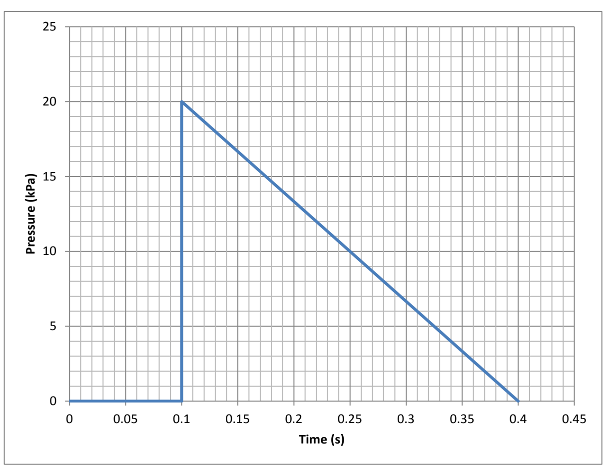


Fig. 2 – Graph of pressure against time due to the Design Basis external explosion

- Design Basis Event (DBE) Seismic Loads, a response spectrum scaled to a 0.25g Peak Ground Acceleration (PGA) has been used, Fig. 3. The response spectrum is a combination of the EUR 0.2g and EUR 0.25g spectra for Hard Soil. The vertical spectrum is taken as being equal to 2/3 of the horizontal spectrum. The main structural frame is designed to remain elastic and fully functional following a DBE.

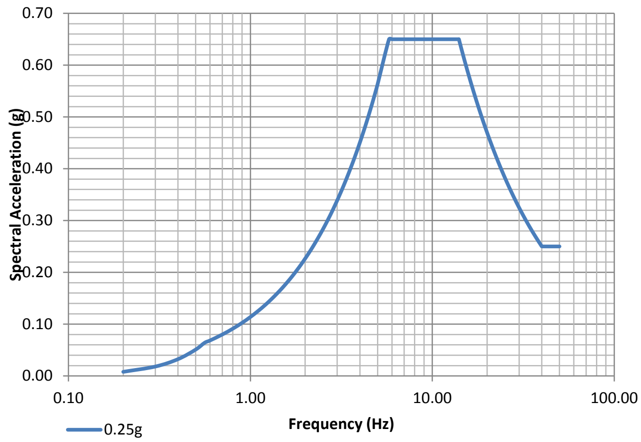


Fig. 3 – 0.25g Horizontal Ground Spectrum with 5% damping

- Beyond Design Basis (BDB) seismic load, a response spectrum scaled to 0.4g peak ground acceleration has been considered, Fig. 4 to evaluate the cliff edge effect.

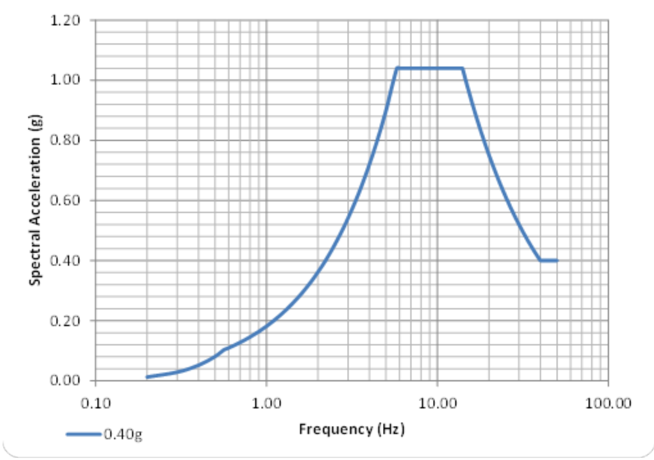


Fig. 4 – 0.4g Horizontal Ground Spectrum with 5% damping

4. Main Structural Steelwork Design Considerations

Generally the main frame steelwork has been analysed and designed using a 3D finite element model constructed in SAP2000 v16, Ref. [1]. The exceptions to this are the local assessments carried out for external explosion, tornado blown missiles and BDB events.

The following design approach has then been adopted:

- Size columns for tornado blown missile and external explosion (local checks)
- Create 3D SAP 2000 model
- Apply Design Basis Events loads to model and set up load combinations
- Complete code based design checks on all members for Design Basis Events
- Adjust member sizes as required
- Perform separate analyses for Beyond Design Basis Events (BDB) including push-over to confirm performance of the structure is acceptable

- Resize members and repeat stages until a satisfactory arrangement has been found. The final scheme efficiently resists Design Basis Events elastically while also providing an acceptable performance for BDB events.

A chevron bracing arrangement has been adopted. This system is more structurally efficient and results in more predictable behaviour post yield: Under BDB loads, the braces have been designed to yield at known locations called “fuses” or “plastic hinges” ensuring failure will occur in a ductile manner.

4.1 Column Assessment for Tornado Generated Missile

Generally, the structural cladding will deform to absorb the energy of the tornado generated missile. However, if a tornado blown missile hits the structural cladding close to a column location, it is assumed that most of the missile’s energy will be absorbed by deformation of the column rather than the structural cladding rail.

The main external columns have been assessed for impact from a tornado blown missile using a non-linear time history analysis: A single column has been modelled in SAP2000 and the critical Automobile missile impact has been applied to determine the column’s non-linear response.

The automobile missile governs the column design. It has been represented as a rectangular pulse load using the guidance given in BS EN 1991-1-7 Ref. [2], as shown in Fig. 5. It is assumed that all of the missile’s energy is absorbed by deformation of the column (soft impact) rather than deformation of the missile. In the SAP2000 analysis, the hinge is assumed to have a ductility of 20 and rotation is limited to 12 degrees in accordance with Blast Effects on Buildings Ref. [3], for a protection category 2. These limits imply extensive plastic deformation of the element and the need for subsequent repair or replacement.

In addition to loading directly from the missile, the stress in the column due to tornado wind pressures has been included in the analysis.

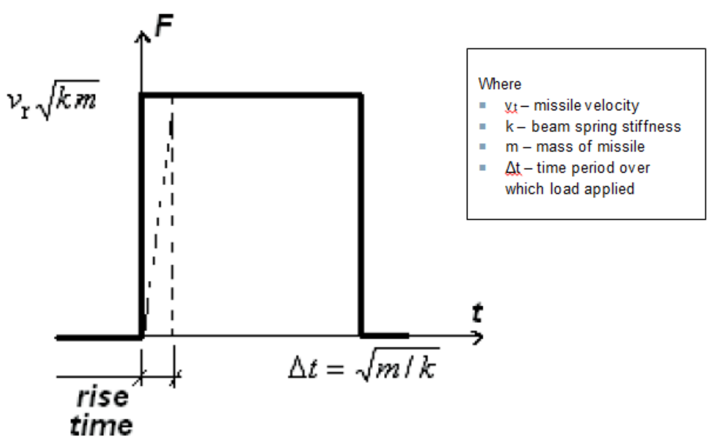


Fig. 5 – Representation of tornado blown missile load

Fig. 6 shows the final deformed shape of a perimeter column having been hit on its major axis by the automobile.

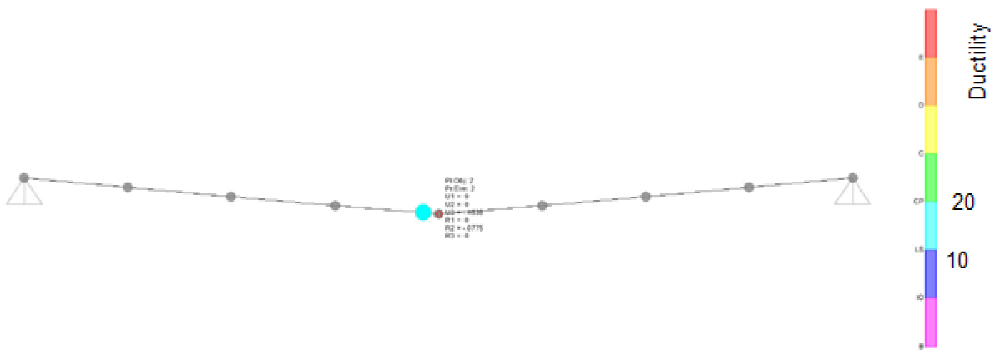


Fig. 6 – Results from SAP2000 for Automobile hitting column showing plastic hinge

For impact on the column's minor axis, a slightly modified approach has been adopted. Due to the support provided by the vertical bracing, the column span is half its overall height. Using the method given in Blast Effects on Buildings Ref. [3], the stiffness of the automobile (= 300 kN/m from Ref. [2]) has been compared to that of the column: The column was found to be significantly stiffer and so deformation of the automobile will be the major source of energy absorption (hard impact):

Using the automobile spring stiffness the equivalent force and duration of the load (see Fig. 5) have been determined and applied in a SAP2000 model. The analysis shows that the column remains elastic, when hit on its minor axis by the tornado blown missile.

4.2 3D Finite Element Model

The main frame steelwork has been modelled in SAP2000. Secondary steelwork such as hot rolled purlins, sheeting rails and steelwork supporting partitions has not been explicitly included in the model. Along with the architectural and structural cladding systems, they are included as a load only.

As the structural form is a braced frame in both directions, end restraints and supports have been modelled as pinned connections. Supports are taken to be rigid for translational movement. Loads have typically been input as line loads applied to the columns or rafters. For the Design Basis seismic assessment these line loads have been converted to the equivalent line masses.

Design checks to BS EN 1993-1-1 Ref. [4] have been carried out for all load combinations using the design routine available within SAP2000.

4.3 Design Basis Seismic Assessment (10^{-4} Seismic Event)

For the design basis seismic event (0.25g horizontal ground motion), a linear elastic response spectrum (modal) analysis has been performed in the SAP2000 model, Fig. 3. Sufficient degrees of freedom have been modelled to capture the major modes and accurately reflect the principal structural response characteristics of the building. Modal combination has been performed using the CQC method while directional combination has utilised the SRSS method.

300 modes have been analysed capturing 100% of the mass in each direction. There is one clear main mode in both the north-south and east-west directions. In the vertical direction mass participation is spread over more modes but the highest levels are for 3 modes around a frequency of 32 hertz. The spectral accelerations for the two main horizontal modes shown in Fig. 7.

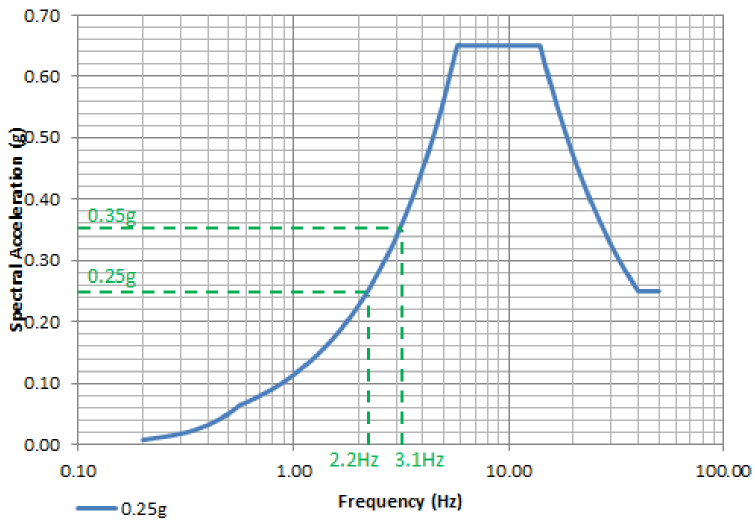


Fig. 7 – Spectral accelerations for main horizontal modes

4.4 Push Over and Time-History Analyses

A number of push-over and time-history analyses have been undertaken to assess the beyond yield behaviour of the building. For the push-over analyses, loads are applied incrementally to a non-linear model of the structure to determine the loads at which plastic hinges form and ultimate collapse occurs. For the time-history analyses a similar non-linear model is used, but a known force varying with time is applied, and the level of yielding and deformation of the structure is assessed.

The main steelwork members of the structure have been selected such that yielding and plastic behaviour will initiate in the braced bays. Given this and the relatively symmetrical layout of the building, the structure can be represented by two 2D models: one representing the bracing resisting the east-west loading and the other representing the bracing resisting the N-S loading.

4.4.1 BDB Seismic Event (0.4g)

The dominant horizontal modes identified by the 3D response spectrum analysis are overall sway modes that have mass participations of 64% and 73%. This demonstrates the validity of representing the building as 2D frames for the pushover analysis. The non-linear models of the bracing have been constructed in SAP2000. Given the relative strengths of the members in the braced bays, plastic hinges have been modelled at the mid-points of the diagonal braces (hinges formed in compression). The hinge properties have been based on the recommendations of ASCE 43-05 Ref. [5], and allow for some strain hardening post yield before a significant loss of strength when a ductility ratio of 2.0 is reached (see Fig. 8).

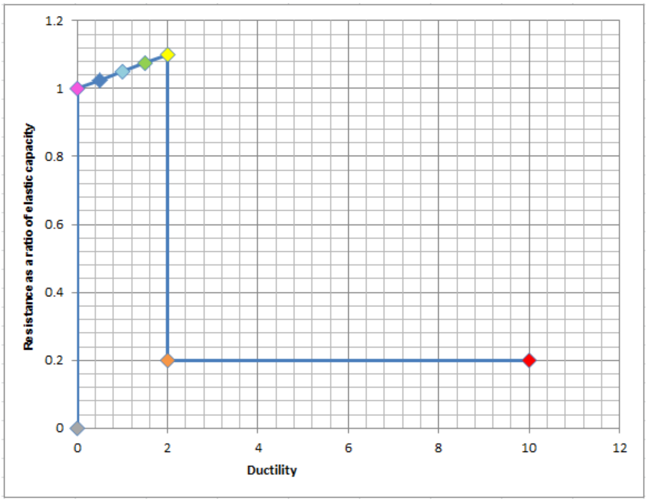


Fig. 8 – Graph showing modelled performance of plastic hinge (μ = 2)

Loads have been applied at the column/tie-beam intersections at the top and middle of the frames. The load at the top is larger than that in the middle, such that the ratio of the loads is equal to the ratio of the deformations at the same locations, determined from the first mode shape of the 2D frame.

Fig. 9 show the plastic hinges at failure. For seismic assessment failure is considered to occur when ductility demand reaches 2 at any of the plastic hinges in accordance with ASCE 43-05 Ref. [5].

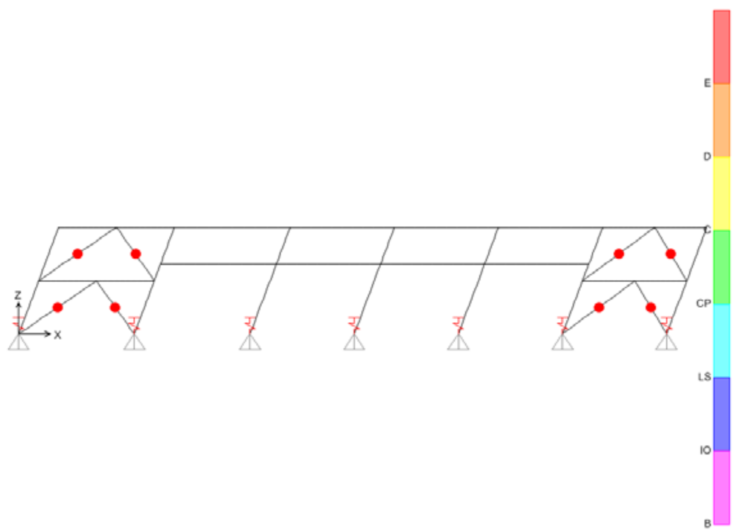


Fig. 9 – Plastic hinges at failure

4.4.2 BDB Tornado (T7) Wind Pressure

The pushover analysis for the T7 tornado has adopted the same basic approach as the seismic pushover model. The incremental forces applied at the top and mid-height of the bracing have been applied in the same proportion as the actual T7 tornado loads. The gradual load application has captured yielding of the members and the development of plastic hinges (fuse points).

From the 3D elastic analysis the total base reactions for the critical north-south braced frame for the most onerous T7 load combination. Comparing this to the horizontal base reactions at failure taken from the push-over analysis it has been demonstrated that the building can withstand the T7 tornado.

4.4.3 External Explosion

The assessment for the external explosion has utilised a non-linear time-history analysis. The applied load has been applied at the top and mid-height of the bracing. These forces have been applied as a pulse load which reduces linearly from the full value to zero over a period of 300ms in line with the specified explosion pressure shown in Fig. 2.

As for the tornado pushover analysis, a non-linear 2D model of the critical N-S bracing has been adopted. However as the explosion loading occurs over a very short time period a greater level of ductility and rotation is acceptable. In accordance with Blast Effects on Buildings Ref. [3], an allowable ductility of 20 and a maximum rotation of 12 degrees have been specified for the plastic hinges (see Fig. 10). Due to the greater deformation permitted at the hinges that form initially, there is greater potential for hinges to form elsewhere. Therefore additional potential hinge locations have been included in the SAP2000 model.

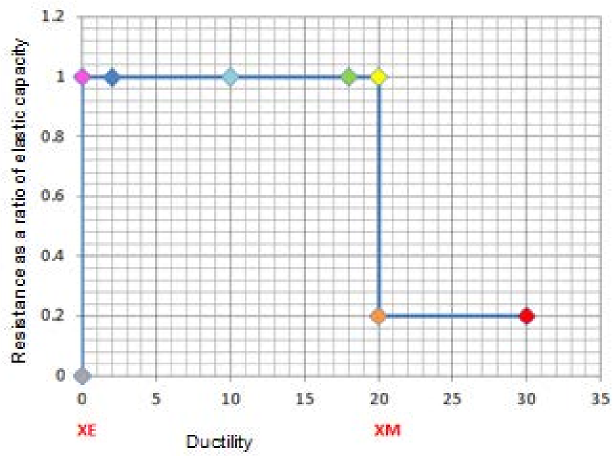


Fig. 10 – Graph showing modelled performance of plastic hinge (μ = 20)

5. Structural Cladding System Design Considerations

5.1 Performance Requirements

The structural cladding system is provided to resist tornado pressures (including differential pressures), tornado blown missiles and pressures resulting from the design basis external explosion.

The main structural frame and structural building envelope has been designed to withstand a Design Basis event with the primary aim to protect the stored equipment inside. Significant damage of the walls and access doors is considered to be acceptable, provided no damage to the internal items is sustained.

It is accepted that architectural cladding may be lost due to wind generated missiles; however, the structural cladding is to remain intact, although plastic deformation of the structural cladding is acceptable. The cladding must not be penetrated by the wind-blown missiles and must absorb the energy of any missile impact or the explosion pressure wave.

5.2 Walls

Two alternative systems were considered for the structural wall cladding:

- Steel plate spanning horizontally on to vertical posts
- Steel plate spanning vertically on to horizontal sheeting rails

Vertical posts would have to resist the applied loads as single span beams. As provision of full strength moment connections at the eaves or base is considered impractical, this would mean that the impact / explosion energy would need to be absorbed by a single plastic hinge. Sensitivity studies investigating this option were undertaken and it was found that very heavy weight sections would be needed to meet the performance requirements, based on this arrangement.

For the alternative system, the span of horizontal rails between columns similar to that of vertical posts: However by making the rails double spanning, two yield points would be formed before failure, enhancing the energy the member can absorb. Due to this superior performance and to avoid providing numerous additional base plates for the vertical post system, it was decided to adopt the cladding system with the horizontal rails and vertically spanning plate.

The tornado blown automobile governs the spacing of the rails: The steel plate is not able to resist the load from the automobile so the rails have had to be spaced such that the automobile would always hit a rail.

5.3 Plate Design

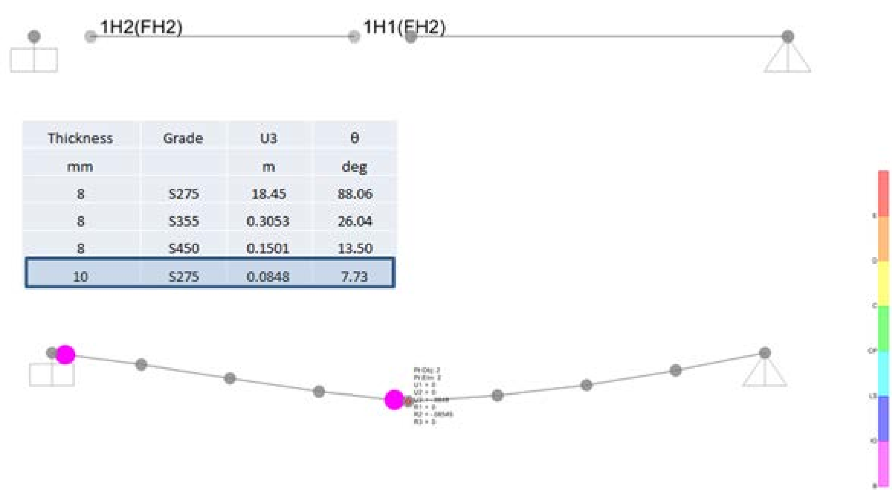
The plate was checked for penetration of tornado generated missiles, using the formula from the Methodology for Tornado Assessment Ref. [6]. This check is independent of the plate span. No account has been taken of the thickness of the architectural cladding as it has not been designed to resist the differential wind pressures due to the tornado so may not be in place to resist the associated missile.

Having established the plate thickness required to prevent penetration an assessment of the plate in bending was then carried out. Given the velocity of the missiles, it has been concluded that any failure would be through penetration of the steel plate rather than bending.

Of the other loads on the plate causing it to bend, the load from the explosion is an order of magnitude greater than the tornado propelled impact. Given this a non-linear dynamic assessment of the plate under the explosion pressure has been carried out to determine the required thickness in bending.

The pressure load on the plate has been determined by taking the product of the reflected pressure and the loaded area of the plate (1m width x span). A SAP2000 model of a simple beam representing the plate has been constructed, and the pressure force on the plate has been applied as a central point load diminishing to zero over 300ms. The plate is considered as one-way spanning and various support conditions have been considered. Plastic hinges have been modelled at mid-span (under the load) and at fixed supports. The hinges are assumed to have a ductility of 20 and rotation is limited to 12 degrees in accordance with Blast Effects on Buildings Ref. [3]. These limits imply extensive plastic deformation of the plate and the need for subsequent repair or replacement.

While a multiple span continuous plate would provide the best performance structurally, it would present difficulties in terms of constructability, due to its size. Instead, a two-span arrangement has been found to be the most favourable. Two hinges will form before failure while the plate size is not excessive. Higher steel grades were investigated, but did not result in a reduction in the required plate thickness (see Fig. 11).



Thickness	Grade	U3	θ
mm		m	deg
8	S275	18.45	88.06
8	S355	0.3053	26.04
8	S450	0.1501	13.50
10	S275	0.0848	7.73

Fig. 11 – SAP2000 model of plate and results

5.4 Structural Cladding Rail Design

The analysis of the structural cladding rail under tornado blown missile load has utilised a similar approach to the assessment of the column: A 2-span beam has been assessed assuming it is hit at mid-span and the collision is treated as a soft impact, with all the energy being absorbed by deformation of the rail rather than the automobile. This is a conservative approach.

One span of the beam is modelled in SAP2000; fixed at one end, to represent the continuous span, and pinned at the other end. Potential plastic hinges have been specified under the load and at the fixed end. The hinge properties assume a ductility of 20 and a rotation limit of 12 degrees Ref. [3].

The automobile missile governs the design. It has been represented as a rectangular pulse load using the guidance given in BS EN 1991-1-7 Ref. [2], as shown in Fig. 5.

The maximum and minimum rail spans have been checked for a variety of section sizes. The deformed shape and plastic hinges predicted following the impact are shown in Fig. 12.

Given the length of the required two-span arrangement for the cladding rails, the member will need to be spliced. A full strength splice will be provided. In addition, a sliding connection will be adopted at one end of the cladding rail to prevent the structural cladding system from acting as a diaphragm.

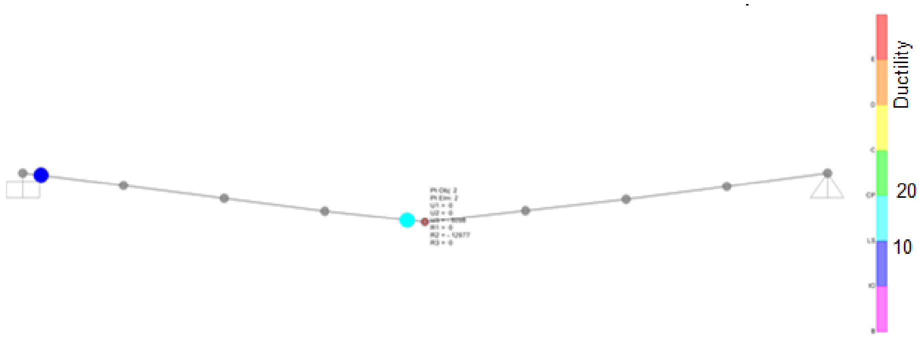


Fig. 12 – Results from SAP2000 for Automobile hitting cladding rail showing plastic hinges

6. Conclusion and Applicability

The specification of highly resilient buildings to resist tornado wind generated missiles in the European market is limited to buildings that are required to perform critical safety functions. Including buildings on Nuclear Sites, where the requirement was identified following the Tohuku event in Japan on the 11th March 2011 and the subsequent impact on the Fukushima-Daiichi station.

The design methodology developed can be tailored to suit alternative client's specific requirements allowing for a broad range of application. They could be applied in other sectors and for other extreme loads where the absorption of energy by the ductile behaviour of the cladding system is required, such as in the Oil and Gas sector where there is potential for extreme blast loads.

7. Acknowledgements

This case study was supported by professional excellence of the nuclear division of Mott MacDonald Limited. We thank our colleagues that participated in the development of the project providing insight and expertise that greatly assisted in the development of the job.

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