

PROGRESSIVE ANALYSIS OF CONTROL ROOM VULNERABILITY

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This paper presents a case study of the cost effective application of progressively sophisticated analytical techniques to an explosion hazard on a large petrochemical facility. The structural response of an existing control room building to the effects of a major vapour cloud explosion was studied using a series of techniques. Starting with a screening level tool and then single and two-degree of freedom analysis, the building performance was checked against international response criteria. Finding that a significant and costly level retrofit was potentially required to meet the postulated loads, the analytical effort was increased to include extensive finite element modelling coupled with invasive material sampling and a further examination of the loads. This paper presents the key findings at each stage of the study, the response of the structure to the loading and the risk reduction and cost benefit reasoning that drove the more increasingly detailed analysis.

1. INTRODUCTION

A control room at a major petrochemical plant in Europe was originally designed in 1979 as a blast resistant structure capable of resisting a design load of 15psi (103kPa) however no specific duration was associated with the load. Following the recent publication of a quantitative risk analysis (QRA) at the plant, the postulated loads on the control room gave cause for concern. Due to the uncertainty associated with the duration of the original design loads, the plant owners requested that BakerRisk Europe Ltd (BakerRisk) undertake a detailed dynamic structural analysis of the control room structure. The aim of the analysis was to identify the response of the major structural components in the control room when subjected to the loads imposed on the building by a vapour cloud explosion. A secondary aim was to identify any structural components that failed to exhibit a satisfactory response, and recommend suitable cost-effective retrofits.

The full analysis of the whole structure was undertaken using single degree of freedom (SDOF) methodology. Local and global responses were considered and a number of potential areas of concern were identified leading to the identification of a number of possible retrofit options. At the request from the client the structure was reanalysed using finite element techniques (FEA) in the hope that a more detailed analysis would remove some of the degree of conservatism inherent in SDOF analysis. Potentially this might have reduced the amount of retrofit necessary or remove the requirement altogether. For the purpose of this paper however, only the analysis of the main roof beams and the associated roof slabs are presented.

1.1 RESPONSE CRITERIA

For a blast resistant design, where the level of protection is described as High, the building response level and number of injuries to building occupants arising from external blast loads to building occupants will be Low, and vice-versa. Note that it should be recognized that is almost impossible to create a truly blast proof structure without committing to an extraordinarily high level of expenditure. For the building considered in this paper, the client did not require a High level of protection and therefore this standard was not considered further. In a blast resistant structure, key structural components are permitted to form plastic hinges as ductility ratios exceed unity,[1] and the level of protection is described as Low to Medium. In the event of a vapour cloud explosion, key structural members are likely to undergo plastic deformation, non-structural elements may possibly fail, and there will be an increased risk of injury to building occupants. The relevant response criteria are summerised below[2,3]

- **Low Response** – Localised building/component damage. The building can be used however repairs are required to restore the integrity of the structural envelope. Total cost of repairs is moderate.
- **Medium Response** – Widespread building/component damage. Building cannot be used until it is repaired. The cost of repairs is significant.
- **High Response** – Building/component has lost structural integrity and may collapse due to environmental conditions (i.e. wind, snow and rain). The cost of repairs approaches the replacement cost of the building.

1.2 SUPPORT ROTATIONS

It is important to note the distinction made between the maximum support rotations for individual structural elements and those recommended for frame members which may on initial inspection appear onerous. However, as the blast wave travels across the structure, the structural framing will be subjected to the large unbalanced horizontal pressure on the front wall and a more conservative response

[1]The ductility ratio (μ) is the ratio of maximum deflection to the limit of elastic deflection.
[2,3] Design of Blast Resistant Buildings in Petrochemical Facilities – ASCE 2010.

Table 1. Component Damage Criteria for Steel Primary Frame Members

Response	Low Response	Medium Response	High Response	Collapse
Support rotation [°]	$0 < \theta \leq 1$	$1 < \theta \leq 1.5$	$1.5 < \theta \leq 2$	$2 < \theta$
Ductility	$0 < \mu \leq 1.5$	$1.5 < \mu \leq 2$	$2 < \mu \leq 3$	$3 < \mu$

Table 2. Component Damage Criteria for Reinforced Concrete Beams, Slabs, and Wall Panels with No Shear Reinforcement

Response	Low Response	Medium Response	High Response	Collapse
Support rotation [°]	$0 < \theta \leq 1$	$1 < \theta \leq 2$	$2 < \theta \leq 5$	$5 < \theta$

is recommended. The values steel and reinforced concrete members are shown in Tables 1 and 2 below[3].

1.3 BLAST LOADS

The control room design loads extracted from the QRA are shown schematically in Figure 1. The figures are in given in millibar and represent a recurrence frequency of 1 in 10^{-4} years. The pressure-time history of the blast waves is assumed to be the shape of a right triangle, which rises instantly to a peak and decays linearly to ambient pressure over the load duration. Load durations were generally in the range 80–180 msecs depending on location.

2. BUILDING DESCRIPTION

The control room was a single storey steel frame building with precast concrete panel walls and cast in-situ reinforced concrete slab roof. The main roof beams were continuous over spans varying from 5.0 m to 15.0 m and bay spacing was constant at 3.75 m. Overall building height was approximately 5.4 m. The precast concrete wall panels were 250 mm thick with 12mm diameter grade 415N/mm^2 bars at 150 mm centres, both horizontally and longitudinally. The reinforced concrete roof slab was cast in-situ on Hollow Rib 38 permanent formwork. The slab was 150 mm thick and also reinforced with 12 mm diameter grade 415N/mm^2 bars at 150mm centres, both horizontally and longitudinally and was connected to the top flange of the steel roof beams using 10mm square section zig-zag bars or lacing. The concrete was typically grade 40 and the cover to the main reinforcement was 30 mm. The steel frame was generally made from European section HE500B beams and HE240B columns. Initially in the absence of any other information the characteristic steel strength was assumed to be 275N/mm^2. Connections were not moment resisting. The foundations consisted of 430 mm diameter bored piles in pairs in reclaimed estuarine gravel/sand. The reinforced concrete floor slab was cast integrally with the ground beams supported on pile caps. The original construction consisted of 8 bays built circa 1979 and was extended to the west by a further four bays some ten years later. Photographs of the building are shown below in Figures 2 and 3.

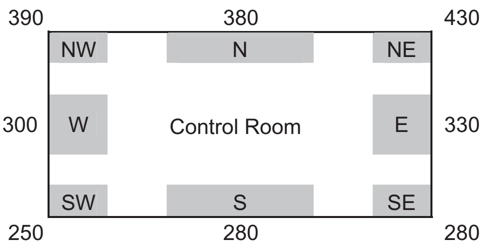


Figure 1. Control Room QRA Free Field Loads *(figures in mbar)*

3. SDOF/TDOF ANALYSIS

Using SDOF/TDOF[4] methodology, the roof beams of the control room were analysed and the results are shown in Table 3.

3.1 SDOF/TDOF ANALYSIS RESULTS

The analysis indicated that most severe support rotation predictably occurred within the 15 m roof beam. The TDOF prediction was 1.4° and corresponded to a Medium Response. At the lowest level of design load (250 mbar, 107 msec), shorter beams had a support rotation of as little as 0.3° which corresponded to a Low Response. As the steel beam in the roof is an effective framing member, the maximum support rotation allowed for a Low Level Response in Table 1 is 1.0°. This therefore posed a dilemma, because one part of the roof met Low Response criteria, whilst another part did not. Clearly there was scope for a further analysis.

4. RESULTS OF FINITE ELEMENT ANALYSIS – PHASE 1

4.1 GENERAL APPROACH

All of the analytical calculations presented in this section utilised high fidelity finite element models. These were used to supplement earlier SDOF and TDOF calculations in order

[4]TDOF – Two Degree of Freedom analysis in which the response of primary and secondary structural elements are analysed compositely.

Figure 2. South Façades

Figure 3. East Façades

to represent some of the geometric and structural complexities of the roof structure. In particular, FEA was needed to represent the composite action between the structural slab and steel roof beam. Accurately modelling the beam using FEA is expected to result in a lower amount of rotation than SDOF and even TDOF analysis. All the calculations were performed using the LS-DYNA nonlinear explicit finite element code. This code was well suited for analyzing transient responses, such as those due to blast loads. The code also included a large library of material models to represent typical structural materials, such as steel and concrete.

4.2 FEA RESULTS

Displacement histories at the mid-point of each span and the corresponding support rotations are shown in Figure 4. The curves do not start at zero because of gravity loading and the maximum response is observed to occur in the longest span of the continuous beam. The peak rotation at this location is 0.65°, which is below the ASCE low response criterion of 1.0° for a primary structural member.

The deformed shape is shown in Figure 5 and has been exaggerated in the vertical direction by a factor of 25. The time selected for the figure corresponds to the peak mid-span deflection in the 15m span. The deformed shape indicates that the concrete slab is not being significantly exercised during the response, but remains relatively rigid. Thus, virtually all the deformation in the roof is due to the deflection of the beam. The exaggerated deformations help to show the nature of the response of the roof beam and the associated concrete slab. Several tensile cracks occur as expected near the mid-spans and at the supports. The most severe cracking is observed on the top surface of the slab in the regions above the column supports, where the maximum negative bending occurs. The crack patterns predicted by the model are similar to those observed in testing and expected of a continuous beam having these boundary conditions. This serves to verify that the model performance is reasonable.

4.3 COMPARISON OF RESULTS (SDOF/TDOF) AND FEA PHASE 1

A comparison of the results obtained using SDOF/TDOF and FEA Phase 1 is shown in Table 4.

Table 3. Results of SDOF/TDOF Analysis

Structural Element	Θ^5	μ^6	X^7_{max}	Remarks
5m Roof Beam	0.3	1.07	24	No retrofit required – Low level of response
15m Roof Beam	1.4	3.6	182	Medium level of response – retrofit required or further detailed study of the design loads and/ or an FE analysis of the roof
9 m & 12m Roof Beams	0.6	1.84	67	No retrofit required – Low level of response

[5]Θ – the angle of support rotation
[6]μ – the ration of elastic to plastic deflection
[7]X_{max} – the maximum deflection in millimeters at the centre of the structural element

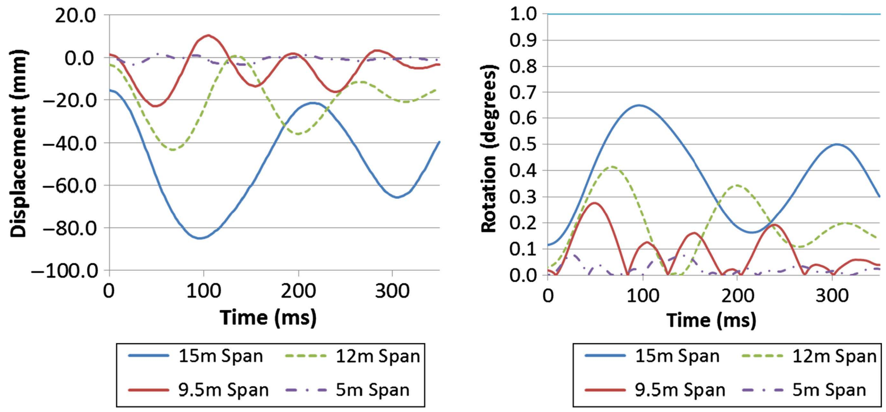


Figure 4. Roof Beam Mid-point Displacement Histories (mm and θ°)

The results of the FEA Phase 1 analysis indicate that more accurate and less conservative estimates of roof framing response can be obtained using a more sophisticated modelling approach. Using high fidelity nonlinear models of the roof beam and roof slab, the analyses demonstrated that by accounting for the postulated composite action between the beam and the roof slab, as well as the added mass of a topping layer of concrete, the beam response would be reduced to below the prescribed criteria. This would eliminate need for structurally upgrading the roof framing.

5. FINITE ELEMENT ANALYSIS – PHASE 2

5.1 GENERAL APPROACH

Following discussions with the client, a decision was taken initiate a second phase of FE analysis to investigate the contribution made by the zig-zag bars to the composite action between the slab and the roof beam. At the same time, the blast loads on the structure were reduced from 430 mbar to 400 mbar (Figure 1) and for the first time a negative phase was incorporated into the blast load pressure time history. The assumption being that the maximum negative pressure was half that of the positive, but for twice the duration as shown in Figure 6.

The characteristic strength of the steel beams was reduced from 275 MPa to 235 MPa as it was felt that without documentary evidence to the former the adoption of a more conservative figure was prudent. Additionally, the model was used to extract more detailed information about axial loads on columns and interface forces between the beam and the structural slab.

5.2 RESULTS OF PHASE 2 ANALYSIS

Histories of the vertical displacement at the centre of each span are shown in Figure 7. The time scale begins when the blast load is first applied to the upstream edge of the roof; the non-zero initial displacements are those associated with gravity. The difference in peak response is trivial, although the application of negative phase loads does somewhat reduce the late-time residual deformations. The widely varying frequencies present in the four different spans should be noted, as one may expect given their different dimensions and (in one case) section properties. Span 4 (5m) barely responds at all, as expected.

To compare the calculated responses to the damage criteria defined earlier (Tables 1 and 2), the peak mid-span displacement is converted to a support rotation, assuming a triangular deformed shape (consistent with plastic response) and taking the arctangent of the deflection divided by half the span. In this way, the support rotations shown in Figure 8 were calculated and are compared to the Low Response criteria of 1.0° rotation. As the figure indicates, the 15 m span exceeds the criterion by a significant margin. Interestingly, inclusion of the negative phase loads actually results in a slight increase in the span 2 peak rotations. This is due to the continuous nature of the beam, with the upward load on one span increasing the moment at the support and causing an increase in the downward

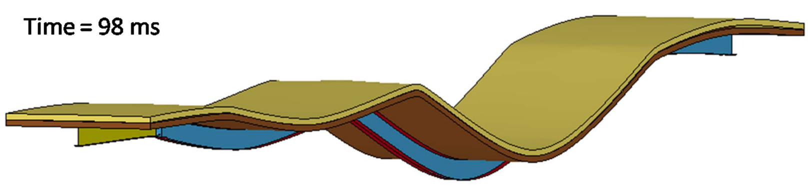


Figure 5. Deformed Shape (exaggerated 25x)

Table 4. Comparison of Results

Structural Element	SDOF/TDOF Θ (°)	SDOF/TDOF X_{max} (mm)	FEA Θ (°)	FEA X_{max} (mm)
Roof beam (span 9.5m)	0.3	24	0.27	22
Roof beam (span 15m)	1.4	182	0.65	84
Roof beam (span 12m)	0.6	67	0.41	42

displacement in an adjacent span. The support rotations in the concrete slab are seen to be far below the allowable criteria and therefore acceptable.

From these results, we conclude that whilst the concrete roof slab response is acceptable, the response of the steel beam is not, especially that of the longest span. Some retrofit will be needed to enhance the strength and stiffness of this portion of the beam in order to reduce its response to an acceptable level that meets the ASCE "Low Response" criteria.

6. FINITE ELEMENT ANALYSIS – PHASE 3

6.1 GENERAL APPROACH

Given that the roof beams required upgrading and that the model was sensitive to material strength, it was paramount that the exact value was discovered. Accordingly, a number of material samples were taken from the webs of the roof beams and tested. At the same time the free field blast loads on the structure were again redefined leading to a reduction of the maximum peak pressure of 363 mbar (down from 400 mbar). The negative phase was retained in the analysis.

Tests undertaken on the steel samples revealed a number of interesting facts depending on where in the building the sample had been taken from. The original control room had been built in 1979, and had been extended 10 years later. The construction drawings indicated that the roof beams in both the original control room and the extension were identical, but examination of the test data showed that two different qualities of steel were present.

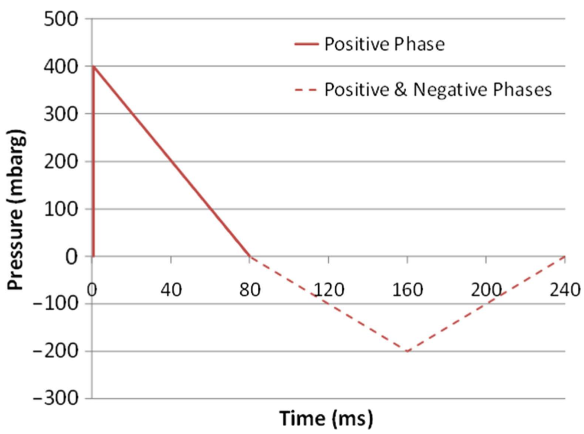


Figure 6. FEA Phase 2 Revised Free Field Blast Load

6.2 ROOF BEAM RESPONSE

The analyses were rerun using both the new and old loads but taking into account the changes in material properties identified in the sample testing. The steel used in extension the proved to be stiffer and stronger, allowing a maximum deflection of about 80mm compared to about 125 mm in the old part of the control room. The change in material properties (relative to those used in earlier studies) is of far greater significance than the change in the loading and the effects are illustrated in Figure 9 and summarised in Table 5.

6.3 ZIG-ZAG BAR RESPONSE

Now that the performance of the roof beams in both parts of the control room, had been shown to exhibit a Low Response without the need for additional reinforcement, it was time to assess the composite action between the steel roof beams and the concrete slab provided by the zig-zag bars (Section 2.0). Manual calculations were undertaken to determine the tensile and shear capacities of the zig-zag

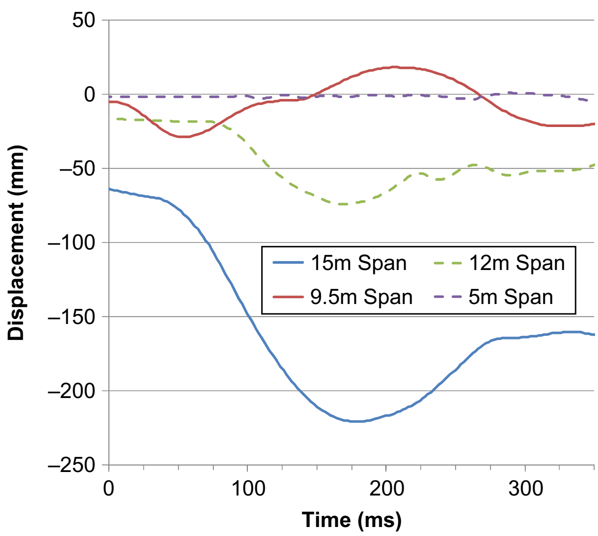
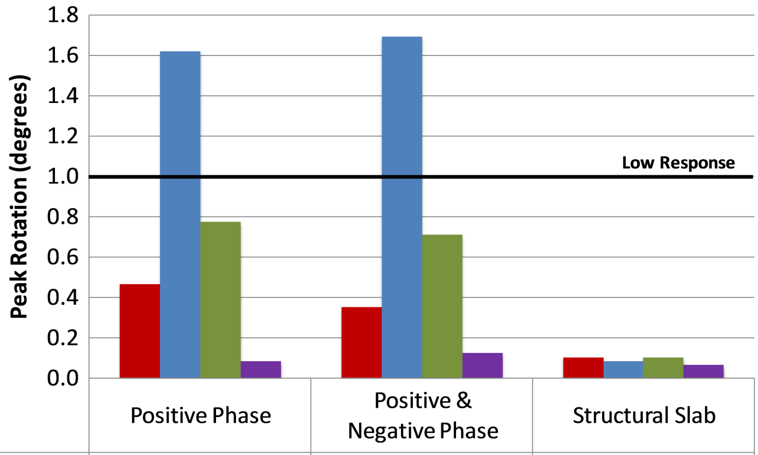


Figure 7. Vertical Displacement at Center of Each Span (Positive and Negative Phase Loads)



	Positive Phase	Positive & Negative Phase	Structural Slab
9.5m Span	0.46	0.35	0.10
15m Span	1.62	1.69	0.08
12m Span	0.77	0.71	0.10
5m Span	0.08	0.12	0.06

Figure 8. Summary of Support Rotations in Beam and Slab

bars and were compared with the results predicted in the FE model and the output is shown in Figure 10.

The graphs show that the peak tensile force (left graph: 100 kN/m at 185 msecs) in the zig-zag bar is well below the calculated capacity of the bar (214 kN/m) however, the shear capacity of the bar is exceeded (right graph: 300 kN/m at 85 msecs). The weld connecting the bar to the steel beam flange is expected to remain intact. In localised regions directly above the columns, the attachments between the structural slab and the steel beam are expected to fail in shear due to failure of the zig-zag bars themselves. This puts into jeopardy the conclusion drawn directly above regarding the adequacy of the beam's flexural response, since that conclusion was dependent on the assumption that full composite action between the structural slab and the steel beam existed. Consequently, more detailed analysis of beam response is needed to assess the impact of these attachment failures on the response of the beam.

6.4 BEAM-SLAB ATTACHMENT FAILURE

The FE model used for this portion of the analyses modified the previous model by eliminating the composite action between the slab and beam. The roof model was then exercised, using the combined positive and negative phase loads since inclusion of the negative phase will exacerbate the propensity for failure of the attachments. The shear force for every attachment along the length of the beam was

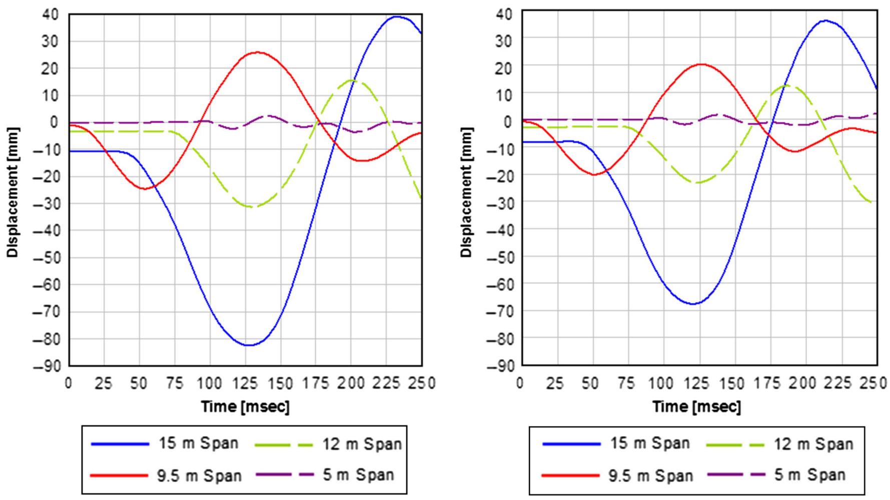


Figure 9. Original (left) and Extension (right) Control Room Beam Deflections (Positive & Negative Phase Loads)

Table 5. Summary of Mid-Span Deflections and Rotations (new building figures in red)

	Old Materials, New Load		New Materials, New Load	
Span	Displacement (mm)	Rotation (°)	Displacement (mm)	Rotation (°)
15 m	−125	0.96	−82 (−68)	0.63 (0.52)
12 m	−45	0.43	−31 (−31)	0.30 (0.30)
9.5 m	−28	0.34	−25 (−20)	0.30 (0.24)
5 m	−3.2	0.07	−3.6 (−2.1)	0.08 (0.05)

analysed to determine all of the failure locations. Results indicated that some 84% of the attachments were predicted to fail in unison with the movement of the blast wave across the roof. The welds that did not fail were located around the middle of each span.

The model was then rerun using a very conservative assumption that the structural concrete slab sits on the beam and provides mass to the system, but the flexural strength is limited to that of the steel beam alone. The results of these final analyses are shown in Figures 11

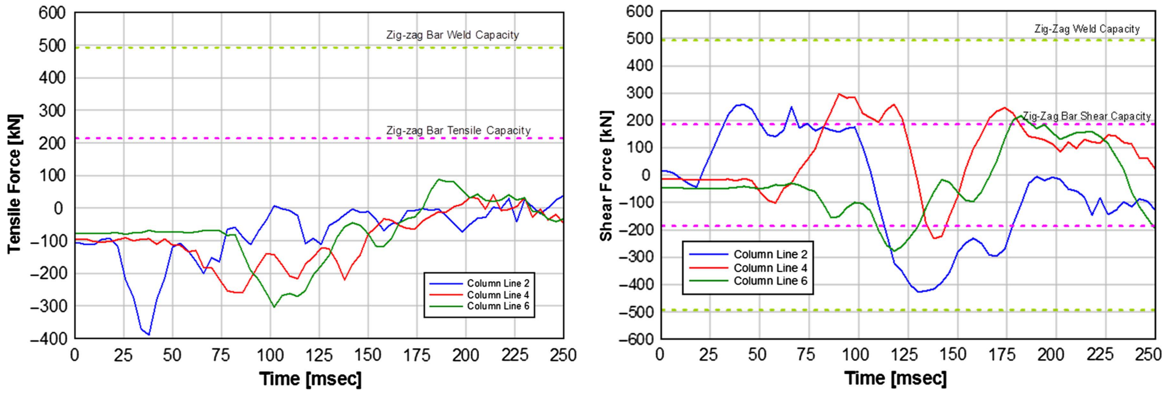


Figure 10. Tensile (left) and Shear (right) Forces in the Zig-Zag Bars

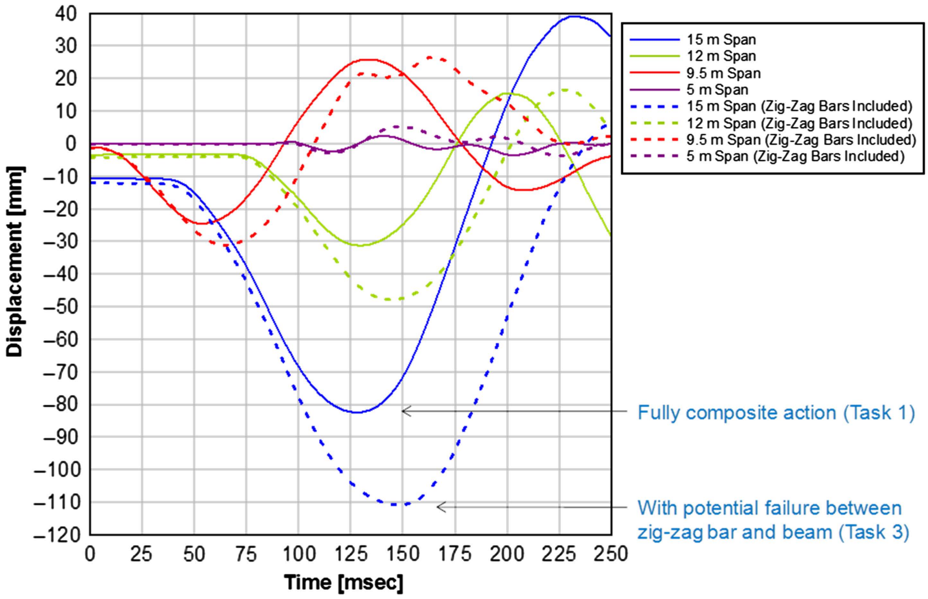


Figure 11. Roof Beam Deflections Allowing for Failure of the Zig-Zag Bars

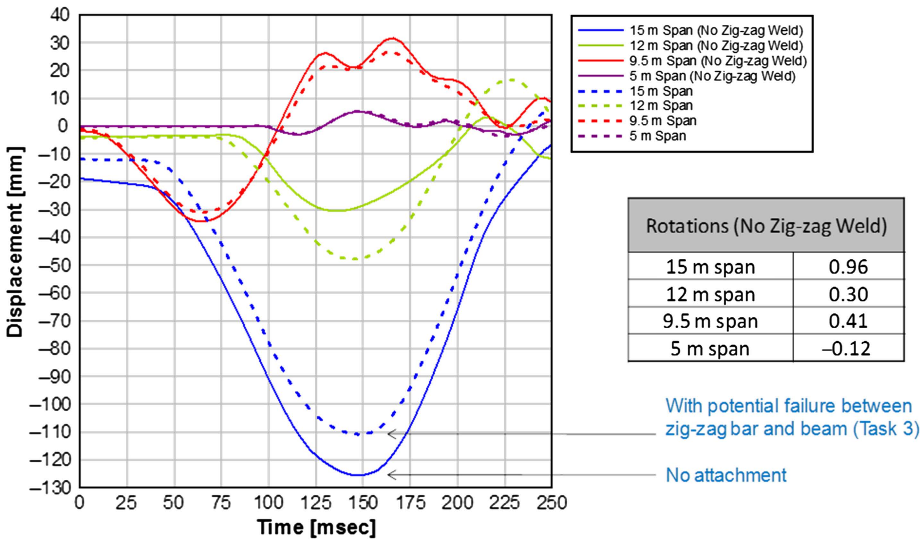


Figure 12. Roof Beam Deflections Assuming No Composite Action

and 12. The graph in Figure 11 shows the roof beam deflections allowing for failure of the zig-zag bars; whilst the graph in Figure 12 shows the beam deflections assuming that the roof slab acts as a dead weight alone (no composite action).

7. SUMMARY

- A majority of the roof slab-to-beam attachments are predicted to fail, in both the original and extension portions of the building.
- Even without full composite action between the slab and the beam, the response of the beam is acceptable and remains below the criterion of 1.0°. However, the consequence of this failure is negligible.
- No retrofits are needed for the roof, either to the beams or the slab-to-beam attachments. Even at Low Response, the building is expected to experience some plastic deformation and need repairs. However, the building functionality is not impaired if roof slab to beam attachments fail. These failures can be repaired post-event if desired.