A Study on Blast Resistance Design of Steel Structures

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Abstract— Process plants in petrochemical industry handle hydrocarbons and other fuels that can produce accidental explosions. Design basis for such plant essentially help to reduce the occurrence of such incidents. Although such incidents may be relatively rare, but the consequences after occurrence such rare but disastrous event can be extremely severe involving personnel causalities, financial losses and potential impacting public safety. In some instances the consequences have involved plant buildings. The concentration of such facilities in buildings points the need of design plant buildings to withstand explosion effects in order to protect the people inside so that, the building does not pose an added hazard to the occupants. In addition to that, it is important to consider blast resistance for certain critical buildings such as control centers & warehouses even if unoccupied, to minimize the impact of accidental explosions on plant operations.

In this paper focus is given on structural aspects of designing or evaluating steel buildings for blast resistance i.e. Design of Blast-Resistant Buildings in Petrochemical Facilities [1]. This paper presents the design aspects of a steel warehouse building for blast loading and its effect on the high response limits of the steel components like column, rafter, purlin, girt & sheeting and bracings. Study on base plate, anchor bolt and foundation design are excluded in present work.

Keywords— American Society of Civil Engineers, Blast Resistance Design, Low-Response Limits, Steel Structures, Warehouse.

I. INTRODUCTION

The analysis and design of structural steel building once blast loading is defined has been discussed in this section of paper from blast resistance point of view. It provides the basic design considerations, principles, procedures and details involved in steel structural design [7, 8 and 9] and evaluation of buildings for blast overpressure effects. The primary objectives for providing blast resistant design for buildings are:

a) Personnel Safety;
b) Controlled shutdown; and
c) Financial considerations.

Table 1

<table>
<thead>
<tr>
<th>Component Response</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
</tr>
<tr>
<td>Medium</td>
</tr>
<tr>
<td>High</td>
</tr>
</tbody>
</table>

Component has none to slight visible permanent damage
Component has some permanent deflection. It Is generally repairable, if necessary, although replacement may be more economical and aesthetic
Component has not failed, but it has significant permanent deflections causing it to be unrepairable

In practice the average yield strength of steel material being installed is approximately 25% greater than the specified minimum values. A strength increase factor (SIF) as shown in Table.II [1] is applied to account for this condition.

Table 2

<table>
<thead>
<tr>
<th>Material</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural Steel (Fy &lt;= 50Ksi or 345Mpa)</td>
</tr>
<tr>
<td>Reinforcing Steel (Fy &lt;= 60Ksi or 414Mpa)</td>
</tr>
<tr>
<td>Cold-Formed Steel</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Material</th>
<th>SIF</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural Steel</td>
<td>1.1</td>
</tr>
<tr>
<td>Reinforcing Steel</td>
<td>1.1</td>
</tr>
<tr>
<td>Cold-Formed Steel</td>
<td>1.21</td>
</tr>
</tbody>
</table>
To incorporate the effect of material strength increase with strain rate, a dynamic increase factor (DIF) as shown in Table III [1] is applied to static strength.

<table>
<thead>
<tr>
<th>Material</th>
<th>DIF</th>
<th>Yield Stress (Fdy / Fy)</th>
<th>Ultimate Stress (Fdu / Fy)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASTM A36</td>
<td>1.29</td>
<td>1.19</td>
<td>1.10</td>
</tr>
<tr>
<td>ASTM A588</td>
<td>1.19</td>
<td>1.12</td>
<td>1.05</td>
</tr>
<tr>
<td>ASTM A514</td>
<td>1.09</td>
<td>1.05</td>
<td>1.00</td>
</tr>
<tr>
<td>ASTM A653</td>
<td>1.10</td>
<td>1.10</td>
<td>1.00</td>
</tr>
</tbody>
</table>

The structures performance goal becomes an important factor in selection of maximum response values. Response deformation limits are used to ensure that adequate response to blast loads is provided. These limits are based on the type of structure or component, are based on the type of structure or component, Construction materials used location of the structure and desired protection level. The response limits for steel components are as shown in Table V [1].

<table>
<thead>
<tr>
<th>Component</th>
<th>Low</th>
<th>Medium</th>
<th>High</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hot Rolled steel compact secondary members (Beams, Girts, Purlins)</td>
<td>3</td>
<td>10</td>
<td>6</td>
</tr>
<tr>
<td>Steel Primary Frame members (with significant compression)</td>
<td>1.5</td>
<td>2</td>
<td>1.5</td>
</tr>
<tr>
<td>Steel Primary Frame members (without significant compression)</td>
<td>1.5</td>
<td>3</td>
<td>2</td>
</tr>
<tr>
<td>Cold-Formed light gage steel beams, Girts, Purlins and Non-Compact secondary hot rolled members</td>
<td>2</td>
<td>1.5</td>
<td>3</td>
</tr>
<tr>
<td>Cold-Formed light gage steel panels</td>
<td>1.75</td>
<td>1.25</td>
<td>3</td>
</tr>
</tbody>
</table>

The basic analytical model used in most blast design applications is the single degree of freedom (SDOF). The equivalent Mass, Stiffness and Loading is obtained through the use of transformation factors as shown in Table IV [1].

<table>
<thead>
<tr>
<th>Loading Diagram</th>
<th>Strain Range</th>
<th>Load Factor, K_L</th>
<th>Lumped Mass Factor, K_M1</th>
<th>Uniform Mass Factor, K_M2</th>
<th>Bending Resistance, R_b</th>
<th>Spring Constant, K</th>
</tr>
</thead>
<tbody>
<tr>
<td>Simply Supported Beam with UDL</td>
<td>Elastic Plastic</td>
<td>0.64</td>
<td>0.50</td>
<td>---</td>
<td>0.50</td>
<td>8 Mpc / L</td>
</tr>
<tr>
<td>Simply Supported Beam with Point Load at Centre</td>
<td>Elastic Plastic</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>0.49</td>
<td>4 Mpc / L</td>
</tr>
<tr>
<td>Fixed Beam with UDL</td>
<td>Elastic Plastic</td>
<td>0.53</td>
<td>0.50</td>
<td>---</td>
<td>0.41</td>
<td>12 Mps / L</td>
</tr>
<tr>
<td>Fixed Beam with Point Load at Centre</td>
<td>Elastic Plastic</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>0.37</td>
<td>4(Mps+Mpc) / L</td>
</tr>
<tr>
<td>Propped Cantilever Beam with UDL</td>
<td>Elastic Plastic</td>
<td>0.58</td>
<td>0.50</td>
<td>---</td>
<td>0.45</td>
<td>8 Mpc / L</td>
</tr>
<tr>
<td>Propped Cantilever Beam with Point Load at Centre</td>
<td>Elastic Plastic</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>0.43</td>
<td>16 Mps / 3L</td>
</tr>
</tbody>
</table>
II. CASE STUDY

In this section a steel ware house building is considered to analyze & design for Low response limits as per ASCE second edition [1]. The dynamic response of the all structural main & secondary elements is calculated by using numerical techniques using linear acceleration method [2, 3, 4, 5 and 6]. Mathcad & Microsoft excel programs are developed to get the non-linear dynamic response of all structural & non structural elements. The stiffness on structural frames is calculated using STAAD Pro. Structural analysis & design software. The Geometry of the structure as given below

Length of the Building = 220.250Ft
Width of the Building = 152.250Ft
Height of the Building = 28.000Ft

And the blast pressures on other side walls and back walls are calculated as per ASCE Code [1].

Table 6  
Blast Loads on the Ware House Building

<table>
<thead>
<tr>
<th>Structure Element</th>
<th>Pressure (Psi)</th>
<th>Impulse (Psi-ms)</th>
<th>Duration (ms)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Worst Wall</td>
<td>1.50</td>
<td>50</td>
<td>67</td>
</tr>
<tr>
<td>Roof</td>
<td>0.80</td>
<td>75</td>
<td>188</td>
</tr>
</tbody>
</table>

Member section sizes and connections shall be designed based on the “Low Response” criteria as published in ASCE [1]. Low response is defined as localized component damage, and a building that can continue to be used with a moderate repair cost to restore the integrity of the structural envelope. The response limits for steel components for the given low response criteria is as mentioned in below

A. Deck Sheet Response (Worst Wall)

Effective Elastic Deformation (Ye) = 3.97mm
Maximum Inelastic Deformation (Ym) = 4.11mm
Actual Ductility Limit = (Ym/Ye) = 1.035 < 1.75 … Ok
Actual Allowable support rotation = 0.687 < 1.25 … Ok

Figure 3 Blast Pressure Diagram on Worst wall (Deck Sheet)

Figure 4 Dynamic Response of the Deck Sheet (Worst wall displacements)
B. Girt Response (Worst Wall)

Effective Elastic Deformation \( Y_e = 68.07 \text{mm} \)
Maximum Inelastic Deformation \( Y_m = 69.09 \text{mm} \)
Actual Ductility Limit = \( \frac{Y_m}{Y_e} = 1.014 < 2.00 \) .... Ok
Actual Allowable support rotation = 1.297 < 1.50 .... Ok

C. Deck Sheet Response (Side & Back)

Effective Elastic Deformation \( Y_e = 3.96 \text{mm} \)
Maximum Inelastic Deformation \( Y_m = 2.23 \text{mm} \)
Actual Ductility Limit = \( \frac{Y_m}{Y_e} = 0.562 < 1.75 \) .... Ok
Actual Allowable support rotation = 0.971 < 1.50 .... Ok
E. Roof Deck Panel Response

Effective Elastic Deformation (Ye) = 7.064mm
Maximum Inelastic Deformation (Ym) = 6.90mm
Actual Ductility Limit = (Ym/Ye) = 0.989 < 1.75 …. Ok
Actual Allowable support rotation = 0.874 < 1.25 …. Ok

F. Roof Purlin Response

Effective Elastic Deformation (Ye) = 68.27mm
Maximum Inelastic Deformation (Ym) = 68.58mm
Actual Ductility Limit = (Ym/Ye) = 1.004 < 2.00 …. Ok
Actual Allowable support rotation = 1.287 < 1.50 …. Ok

G. Main Frame -2

Effective Elastic Deformation (Ye) = 155.20mm
Maximum Inelastic Deformation (Ym) = 112.77mm
Actual Ductility Limit = (Ym/Ye) = 0.728 < 1.50 …. Ok
Actual Allowable support rotation = 0.562 < 1.00 …. Ok
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**H. Main Frame-3, 4 & 5**

![Figure 17 Blast Pressure Diagram on Roof Frame](image)

- Effective Elastic Deformation \((Y_e) = 155.20\) mm
- Maximum Inelastic Deformation \((Y_m) = 165.10\) mm
- Actual Ductility Limit = \((Y_m/Y_e) = 1.262 < 1.50\) .... Ok
- Actual Allowable support rotation = 0.974 < 1.00 .... Ok

![Figure 20 Dynamic Response of the Roof Frame-6&13](image)

**J. Main Frame -7 to 12**

![Figure 18 Dynamic Response of the Roof Frame-3,4&5](image)

![Figure 19 Blast Pressure Diagram on Roof Frame](image)

- Effective Elastic Deformation \((Y_e) = 155.20\) mm
- Maximum Inelastic Deformation \((Y_m) = 80.72\) mm
- Actual Ductility Limit = \((Y_m/Y_e) = 0.575 < 1.50\) .... Ok
- Actual Allowable support rotation = 0.450 < 1.00 .... Ok

![Figure 21 Blast Pressure Diagram on Roof Frame](image)

- Effective Elastic Deformation \((Y_e) = 155.20\) mm
- Maximum Inelastic Deformation \((Y_m) = 161.30\) mm
- Actual Ductility Limit = \((Y_m/Y_e) = 1.020 < 1.50\) .... Ok
- Actual Allowable support rotation = 0.787 < 1.00 .... Ok

![Figure 22 Dynamic Response of the Roof Frame-7 to 12](image)
III. CONCLUSION

From the above case study, it can be concluded that the process plant of petrochemical industry which handles even hydrocarbons and other fuels which produce accidental explosions needs a special attention in analysis and design from the researchers and practicing engineers from consideration of blast resistance. Although the occurrence of explosive incidents may be relatively rare, but keeping in view of the consequences after occurrence such rare but disastrous event focus must be given to the need of blast resistance design of plant buildings (even warehouses) to withstand explosion effects in order to protect the occupants and resources. The paper also focuses the help of non-linear dynamic analysis techniques to find the individual structural element responses in addition to the linear static method.

Note: The results presented in above section are strictly applicable to the warehouse structure considered in case study & assumptions considered herein.

REFERENCES