10. EFFECTS OF BLASTS AND TSUNAMI ON BUILDINGS

10.1 GENERAL

As a result of an explosion, a shock wave is generated in the air which moves outward in all directions from the point of burst with high speed causing time-dependent pressure and suction effects at all points in its way. The shock wave consists of an initial positive pressure followed by a negative (suction) phase at any point. The shock wave is accompanied by blast wind causing dynamic pressures due to drag effects on any obstruction coming in its way. Due to diffraction of the wave at an obstructing surface, reflected pressure is caused instantaneously which clears in a time depending on the extent of obstructing surface [112].

The disasters of Blasts and Tsunami have been examined here in brief so as to cover these two disaster events which have caused considerable havoc in recent times. The effect of Blasts on the structure can be automated and design considerations would definitely impact the performance of the building. Hence Blast Loads and Structural performance have been discussed in a broad framework.

Tsunami effects are more widespread and have more of non-structural aspects to be managed rather than the structural aspects. However the recommendations of various agencies have been compiled here.

10.2 EXPLOSION AND BLAST PHENOMENA

The detonation of a condensed high explosive generates hot gases under pressure up to 300 kilo bar and a temperature of about 3000-4000°C. The hot gas expands forcing out the volume it occupies. As a consequence, a layer of compressed air (blast wave) forms in front of this gas volume containing most of the energy released by the explosion. Blast wave instantaneously increases to a value of pressure above the ambient atmospheric pressure. This is referred to as the side-on overpressure that decays as the shock wave expands outward from the explosion source. After a short time, the pressure behind the front may drop below the ambient pressure (**Fig.10.1**). During such a negative phase, a partial vacuum is created and air is sucked in [113]. This is also accompanied by high suction winds that carry the debris for long distances away from the explosion source.

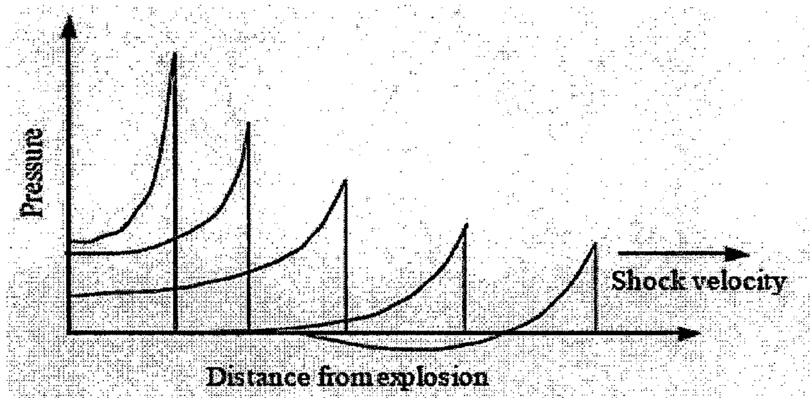


Fig. 10.1 Blast Wave Propagation

At any surface encounterd by the shock wave, the pressure rises almost instantaneously to peak values of overpressure and the dynamic pressure or their reflected pressure. The peak positive intensity quickly drops down to zero; the total duration of the positive phase being a few milliseconds.

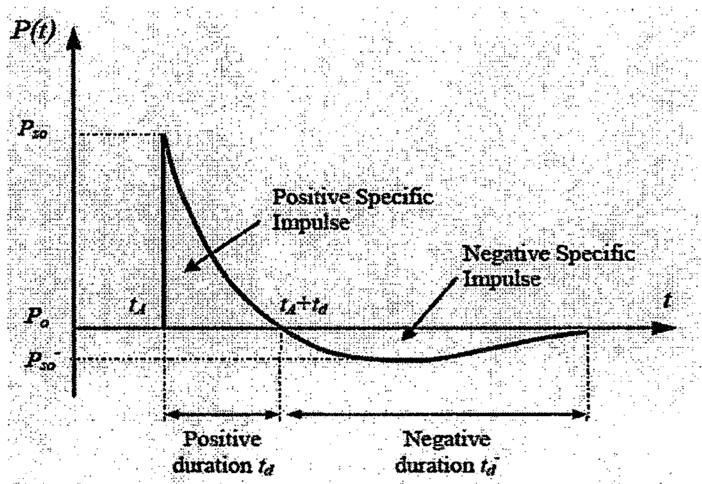


Fig. 10.2 Blast Wave Pressure: Time History

The maximum negative overpressure is much smaller than the peak positive side on overpressure, its limiting value being one atmosphere. But the negative phase duartion is 2 to 5 times as long as that of the positive phase shown in **Fig. 10.2**.

10.3 GENERAL PRINCIPLES OF BLAST LOADING

As in the case of normal loads, members subjected to blast pressures resist the applied force by means of internal stresses developed by them (**Fig. 10.3)**. However, the effective load due to blast, for which resistance should be developed in the member, depends upon the dynamic properties of the member itself. Longer the natural time period of the member smaller is the effective load for design. The duration of the positive phase of the blast is generally small as compared with natural period of the structural elements, hence may be treated as an impulse problem.

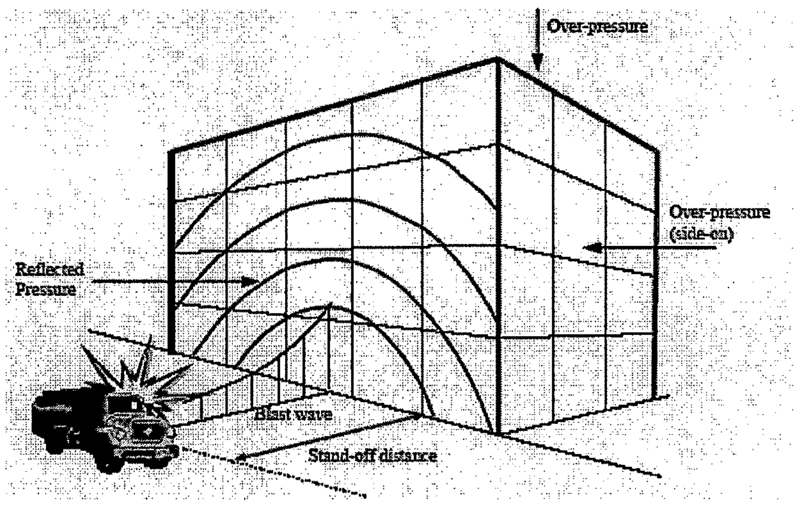


Fig. 10.3 Blast Loads on Building

10.3.1 Blast Wave Scaling Laws

All blast parameters are primarily dependent on the amount of energy released by a detonation in the form of a blast wave and the distance from the explosion. A universal normalized description of the blast effects can be given by scaling distance relative to $(E/Po)^{1/3}$ and scaling pressure relative to Po, where E is the energy release (kJ) and Po the ambient pressure (typically 100 kN/m^2). For convenience, however, it is general practice to express the basic explosive input or charge weight W as an equivalent mass of TNT. Results are then given as a function of the dimensional distance parameter (scaled distance) $Z = R/W^{1/3}$, where R is the actual effective distance from the explosion. W is generally expressed in kilograms. Scaling laws provide parametric correlations between a particular explosion and a standard charge of the same substance.

10.3.2 Prediction Of Blast Pressure

Blast wave parameters for conventional high explosive materials have been the focus of a number of studies during the 1950's and 1960's. Newmark and Hansen (1961) introduced a relationship to calculate the maximum blast overpressure, *Pso*, in bars, for a 2gh explosive charge detonates at the ground surface as:

$$P_{so} = 6784 \frac{W}{R^3} + 93 \left[\frac{W}{R^3}\right]$$

There are mainly two types of structures:

a) Diffraction Type Structures: These are closed structures without openings, with the total area opposing the blast. These are subjected to both the shock wave side on overpressure pa and the dynamic pressure q caused by the blast wind.

b) Drag type Structures: These are the open structures composed of elements like beams, columns, trusses etc. which have small projected area opposing the shock wave. These are mainly subjected to dynamic pressure q.

10.4 RESPONSE OF STRUCTURAL ELEMENTS TO BLASTS

The significant factors, on which the response of a structural element subjected to blast force depends, are the pressure versus time diagram acting on the element, the effective time period of the element, the resistance versus deflection diagram of the element, and the maximum permissible deflection [113]. The maximum permissible deflection defines the energy absorption capacity of the element which is equal to the area under the resistance deflection curve.

Complexity in analyzing the dynamic response of blast-loaded structures involves the effect of high strain rates, the non-linear inelastic material behavior, the uncertainties of blast load calculations and the time-dependent deformations. Therefore, to simplify the analysis, a number of assumptions related to the response of structures and the loads has been proposed and widely accepted. To establish the principles of this analysis, the structure is idealized as a single degree of freedom (SDOF) system and the link between the positive duration of the blast load and the natural period of vibration of the structure is established. This leads to blast load idealization and simplifies the classification of the blast loading regimes.

10.5 FAILURE MODES OF BLAST-LOADED STRUCTURES

Blast loading effects on structural members may produce both local and global responses associated with different failure modes. The type of structural response depends mainly on the loading rate, the orientation of the target with respect to the direction of the blast wave propagation and boundary conditions. The general failure modes associated with blast loading can be flexure, direct shear or punching shear. Local responses are characterized by localized bleaching and spalling, and generally result from the close-in effects of explosions, while global responses are typically manifested as flexural failure.

The global response of structural elements is generally a consequence of transverse (out-of-plane) loads with long exposure time (quasi-static loading) and is usually associated with global membrane (bending) and shear responses. Therefore, the global response of above-ground reinforced concrete structures subjected to blast loading is referred to as membrane/bending failure.

The second global failure mode to be considered is shear failure. It has been found that under the effect of both static and dynamic loading, four types of shear failure can be identified: diagonal tension, diagonal compression, punching shear, and direct (dynamic) shear. The first two types are common in reinforced concrete elements under static loading while punching shear is associated with local shear failure, These shear response mechanisms have relatively minor structural effect in case of blast loading and can be neglected. The fourth type of shear failure is direct (dynamic) shear. This failure mode is primarily associated with transient short duration dynamic loads that result from blast effects, and it depends mainly on the intensity of the pressure waves. The associated shear force is many times higher than the shear force associated with flexural failure modes. The high shear stresses may lead to direct global shear failure and it may occur very early (within a few milliseconds of shock wave arrival to the frontal surface of the structure) which can be prior to any occurrence of significant bending deformations.

10.6 SOFTWARE FOR BLAST EFFECTS

Computational methods in the area of blast effects mitigation are generally divided into those used for prediction of blast loads on the structure and those for calculation of structural response to the loads. Computational programs for blast prediction and structural response use both first-principle and semi-empirical methods. Programs using the first principle method can be categorized into uncouple and couple analyses. The uncouple analysis calculates blast loads as if the structure (and its components) were rigid and then applying these loads to a responding model of the structure. The shortcoming of this procedure is that when the blast field is obtained with a rigid model of the structure, the loads on the structure are often over-predicted, particularly if significant motion or failure of the structure occurs during the loading period. For a coupled analysis, the blast simulation module is linked with the structural response module. In this type of analysis the CFD (computational fluid mechanics) model for blast-load prediction is solved simultaneously with the CSM (computational solid mechanics) model for structural response. By accounting for the motion of the structure while the blast calculation proceeds, the pressures that arise due to motion and failure of the structure can be predicted more accurately. Examples of the software which supports this type of analysis facility are AUTODYN, DYNA3D, LSDYNA and ABAQUS.

10.7 PROGRESSIVE COLLAPSE ANALYSIS - CASE STUDY

A number of European countries, USA and Canada have incorporated progressive collapse provisions in their building codes. ANSI recommends the alternative path method, in which the local failure is allowed to occur but an alternative path must be provided around the failed structural elements.

A 52 storey building (modified from a typical tall building designed in Australia) was analyzed in this study. The plan view and structural configuration of the building are shown in **Fig. 10.4**. The typical story height is 3.85 m. Perimeter columns are spaced at typical 8.4 m centers and are connected by spandrel beams to support the facade. The lateral loads are resisted by 6 core boxes located at the centre of the structural plan. The building is designed to resist lateral loads due to wind and seismic ground motion. The slab, columns and core walls are all cast-in-place concrete. The lateral load resistance system of the building relies mainly on the lateral load capacity of the core walls which account for about 80% of the overall capacity.

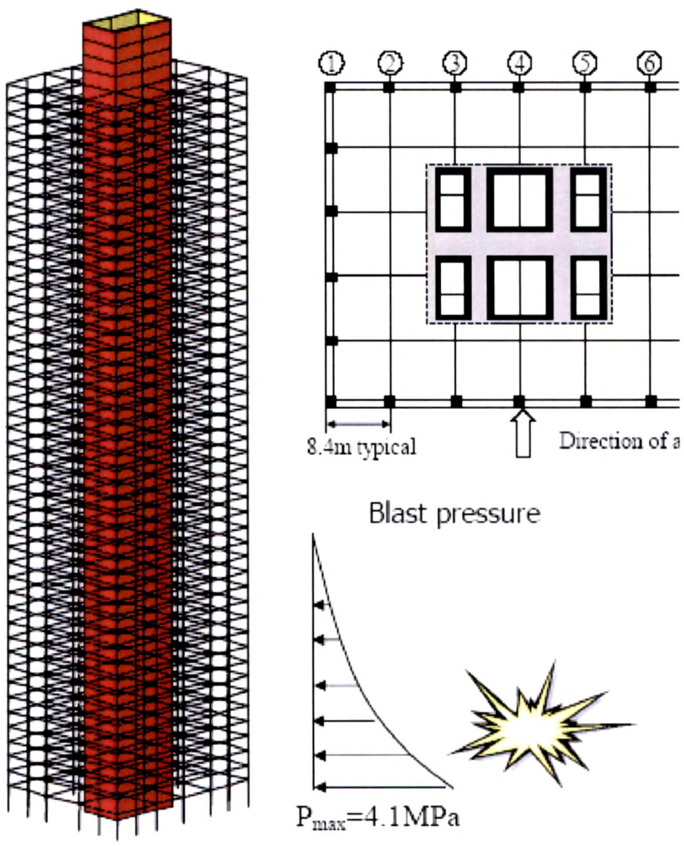


Fig. 10.4 Structural Configuration

Figures 10.5 and **10.6** show the effect of direct blast pressure on perimeter columns, beams and floor slabs. The concrete slabs in this example building are 125 mm thick supported by prestressed wide band beams. The pressure below the slab is greater than the pressure above and causes the net upward load on each slab (**Fig. 10.6**). To detect local damage, the blast analysis was carried out for perimeter columns, beams and floor slabs based on the actual blast pressures on each element. Results plotted in **Fig. 10.7** show column lines 4, 5 of the ground and 1st levels failed due to the direct impact of the blast wave. Slabs and beams from column line 3 to 6 also collapsed.

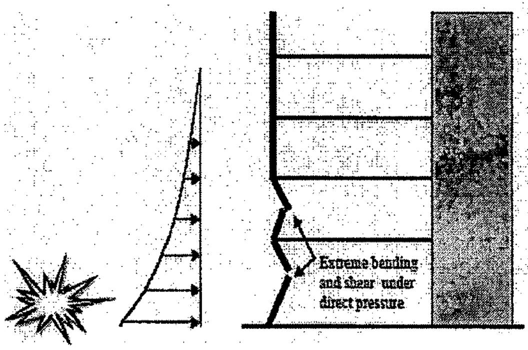


Fig. 10.5 Direct Column Blast Pressure

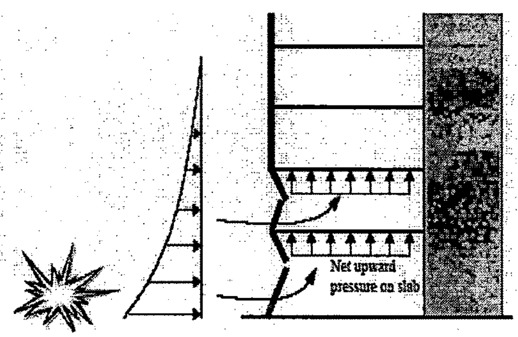
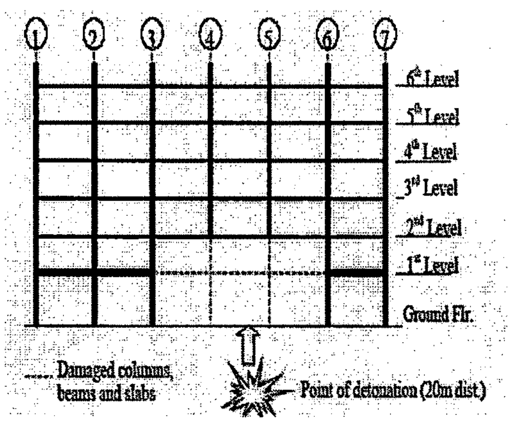


Fig. 10.6 Uplifting Floor Slab due to Blast

As seen from **Fig. 8**, the alternative load paths go through columns surrounding the damaged area where the vertical loads are transferred. Beams and floor slabs above that area become critical due to the loss of the supporting columns. The overall stability of the structure will rely on continuity and ductility of these elements to redistribute forces within the structure. The falling debris of the collapsed members also imposes severe loading on the floors below. It is essential to check whether that overload can be carried without causing further collapse.



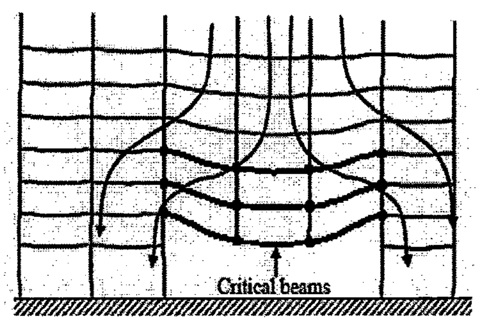


Fig. 10.7 Damaged Columns and Beams **Fig. 10.8 Progresive Collapse Analysis**

10.8 WTC-FEMA EXECUTIVE SUMMARY

Following the collapse of the World Trade Centres, a combined study by FEMA and SCI-ASCE was undertaken [114]. Following observations were made:

The structural damage sustained by the two buildings as a result of the terrorist attacks was massive. The fact that the structures were able to take this level of damage and remain standing for an extended period of time is remarkable. Events of this type, resulting in such substantial damage, are generally not considered in building design. Preliminary analysis of the damaged structures, suggests that in the absence of further severe loading such as an earthquake or windstorm, they would have remained in their position, However the structures were subjected to a second simultaneous load of the fires caused by the aircraft impacts. The large quantity of jet fuel carried by the aircraft ignited upon impact and caused a series of fires successively on many floors. Over the period of many minutes the heat induced additional stresses into the damaged structural frames while simultaneously softening and weakening the frames. This additional loading and the resulting damage were sufficient to induce collapse of both the structures.

The issues identified from the study of the damage are summarized as follows:

- Structural framing system needs redundancy, so that alternative paths or additional capacity are available for transmitting loads when a building damage occurs.
- Fireproofing needs to adhere to the structural members even in case of an impact, so that the coatings serve their purpose.
- Connection performance under impact and fire loads need to be improved in design as they become critical components in structural frames.
- Fire protection systems and sprinklers require a reliable and redundant water supply, as any interruption will result in failure.

- Egress systems should be evaluated for their robustness.

Thus Blast Loads along with Fire Loads become a critical combination in initiating progressive failures as seen from both the above case studies.

10.9 TSUNAMI EFFECTS AND REHABILITATION

A powerful earthquake measuring 8.9 on the Richter Scale on the Sumatra, Indonesia caused the Tsunami whose devastating effects caused huge losses to life on the Indian subcontinent. Some major aspects of rehabilitation have been discussed in the following sections [115].

10.9.1 Siting Of Building

10.9.1.1 Essential requirements

- During very high velocity winds, the coastal areas suffer due to storm surge where huge loss of property takes place. Site selection should avoid areas likely to be submerged [115]. It is desirable to locate the site such that it is
    - At least 500m from the shore and
    - +5m above Mean Sea Level
- The sites need to be close to the present settlement. It should preferably be within a distance of one km from the present settlement so as to facilitate the fishers to carry out their economic activities easily.
- Building should be located on stable foundation on soil strata having no susceptibility for liquefaction.
- Terrain category and topography of the site should be assessed and based on these correct orientations of the buildings should be finalized.

10.9.1.2 Desirable requirements

- The area behind a mound or a hillock or behind casuarinas plantation should be preferred in order to provide natural shielding.
- In case of non-availability of high-level natural ground, construction can be done either on raised earthen mounds or on stilts with no masonry or

cross- bracing up to maximum tide level. In such cases the stilts should be strengthened by proper Knee bracings

10.9.2 Planning Aspects

10.9.2.1 Essential Requirements

- For individual buildings, a circular or polygonal plan shape is preferred over rectangle or square plans, but from the viewpoint of functional efficiency square plan form is desirable (**Fig.10.9**).

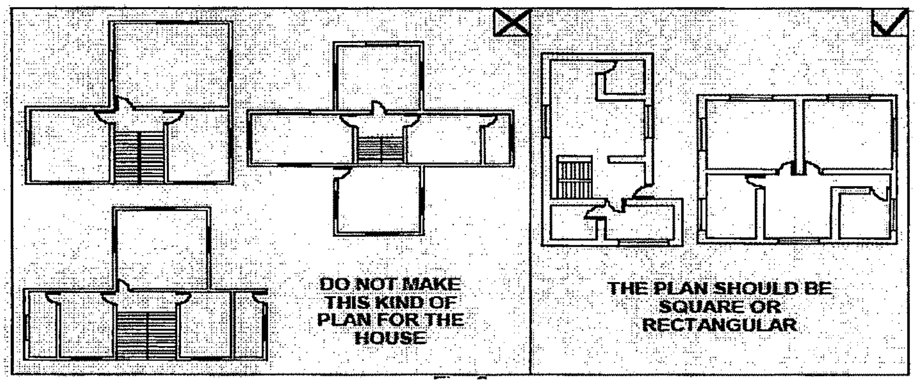


Fig.10.9 Recommended Planning Aspects

- A symmetrical building about both axes with a compact plan-form is more stable than a zigzag plan, having empty pockets. The latter is more prone to wind/ cyclone related damages.
- In case of locating a group of buildings, a cluster arrangement should be chosen in preference to row type buildings (**Fig.10.10**).

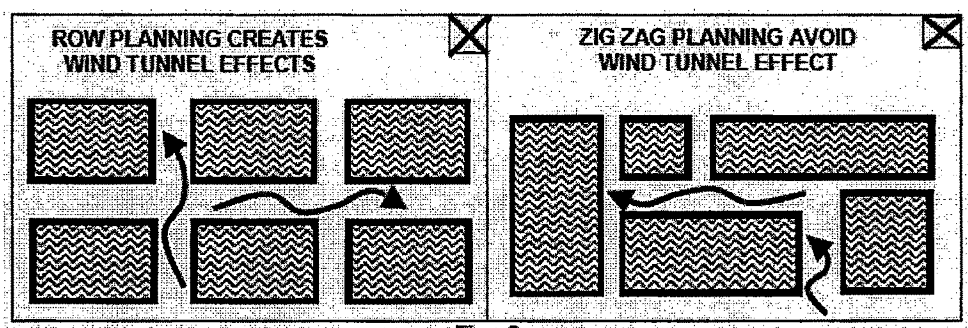


Fig.10.10 Preferred Cluster Arrangement

- The buildings should be oriented in such a manner that the shorter span length of the wall faces the sea.

- Aspect Ratio of the building (Length to Width ratio of the building) should not be greater than 3.
- Ornamental architecture involving large cornices, vertical or horizontal cantilever projection, fascia stones and the like should be avoided.
- The opening in the building will encourage flooding. Hence large openings on the seaside should be avoided.

10.9.2.2 Desirable requirements

- The plan of the building could be square or rectangle from functional point of view. Near circular shape may give the least obstruction to wind or flood waters.
- The individual or twin type units are to be adopted in preference to multistoried or highrise development to facilitate the day-to-day activities of fishers subject to availability of land.
- The plinth area of each dwelling unit should be about 23 Sq.mt. to 28 Sq.mt.

10.9.3 Foundation

10.9.3.1 Essential requirements

- Buildings can have shallow foundation on stiff sandy soil.
- When there is risk of scouring due to storm surge, minimum depth of foundation 1.5 m below natural ground level may be provided in coastal regions.
- In other regions it can be 1 m. The receding tides have a tendency to scour the soil below the foundation. Hence the soil protection work around the building in the form of a raised ground must be considered.
- It will be desirable to connect the individual reinforced concrete column footings by means of RC beams below natural ground level. These bands will be intersecting at right angles and form an integral housing unit. The ground beam should be in one level and be connected continuously.

- The vertical reinforcement at corners and jambs of doors and windows should emanate from ground level beam.
- Horizontal beam at ground level or plinth level must be provided. It is desirable to provide the same at both the places (**Fig.10.11**)

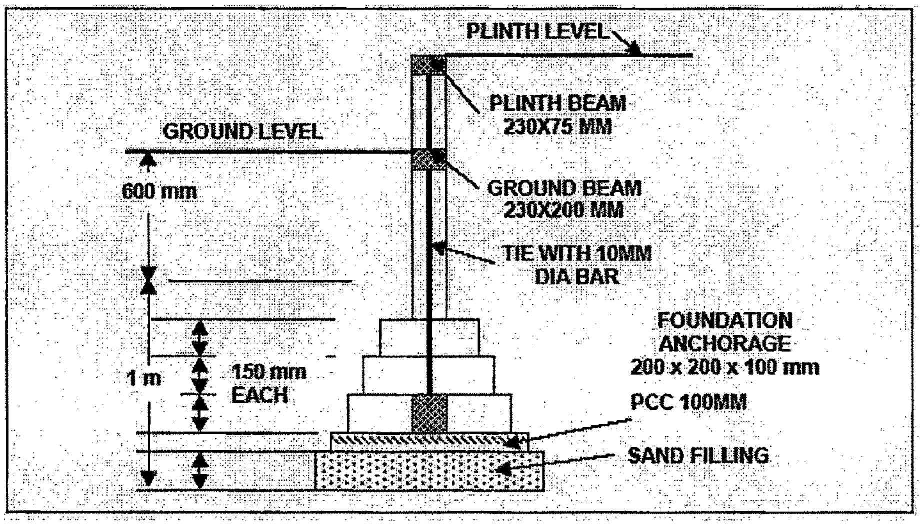


Fig. 10.11 Provision of Tie Beams

- All the vertical bars should have a minimum L bend length of 45.
- The plinth height should be not less than 0.6 m above natural ground level or as per topography requirement.
- In case of strip footing a RC band of 100 mm thickness to the width of the brick masonry should be provided all along the portion and connected to the RC band or slab provided at bottom of foundation of minimum cross section 100 x 100 mm.

10.9.3.2 Desirable

- Continuous reinforced concrete footings are considered to be most effective from earthquake considerations as well as to avoid differential settlements under normal vertical loads and loss of contact during flooding. Hence they are preferable. They offer better resistance to scouring.

- In case of soft soil or filled up soil where the safe bearing capacity near Ground level is very less, pile of single under ream or double under ream should be provided.
- Where the foundation is on rock, it shall be excavated at least to a depth of 50 mm to provide a key for the foundations.

10.9.4 Walling

10.9.4.1 Essential requirements

- All external walls or wall panels must be designed to resist the out of plane lateral pressures adequately. For this, the walls should be sufficiently buttressed by the transverse walls or pilasters. The lateral load due to wind/ tidal surge/ tsunami should finally be resisted either by walls lying parallel to the lateral force direction (by shear wall action) or by RC frames to which the panel walls are fixed using appropriate.
- Reinforcement such as Seismic Bands at window sill and lintel level. This will avoid collapse due to out of plane forces.
- A small building enclosure with properly interconnected walls is preferable. Buildings having long wall should be avoided.
- It is necessary to reinforce walls by means of horizontal reinforced concrete bands and vertical reinforcing tie members having reinforcing bars as suggested for earthquake resistance. This will make the building act as an integral unit under lateral forces (**Fig.10. 12**).

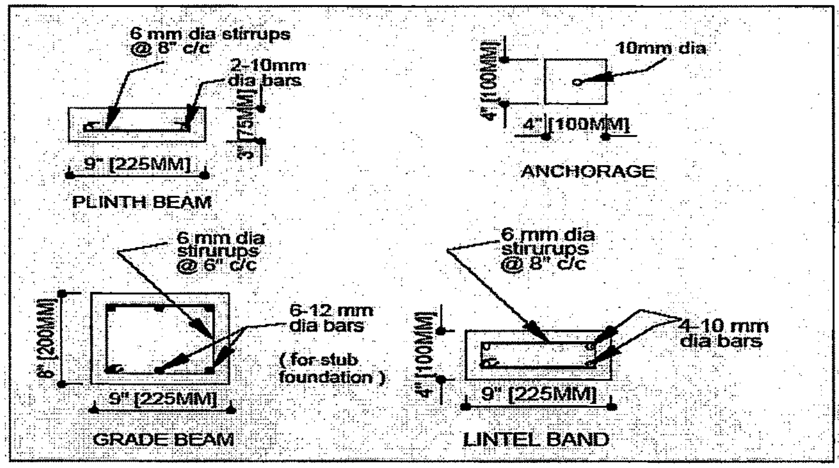


Fig.10.12 Reinforcements in Walls

10.10 SPECIAL STRUCTURAL MEASURES FOR TSUNAMI

Guidelines for general land planning and structures for mitigating the effects of Tsunami are also to be considered as follows [116]:

1. Construction of cyclone shelters.
2. Plantation of mangroves and coastal forests along the coast line.
3. Construction of location specific sea walls and coral reefs in consultation with experts.
4. Development of break waters along the coast to provide necessary cushion against cyclone and tsunami hazards.
5. Development of a "Bio-Shield":A narrow strip of land along coastline. Permanent structures should come up in this zone with strict implementation of suggested norms. Bio-Shield can be developed as coastal zone disaster management sanctuary, which has thick plantation and public spaces for public awareness, dissemination and demonstration.
6. Identification of vulnerable structures and appropriate retrofitting for tsunami/cyclone resistance of all such buildings as well as appropriate planning, designing, construction of new facilities like:
   - ❖ Critical infrastructures e.g. power stations, warehouses, oil and other storage tanks etc. located along the coastline.
   - ❖ All other infrastructure facilities located in the coastal areas.
   - ❖ Public buildings and private houses.

- All marine structures.
- Construction and maintenance of national and state highways and other coastal roads.

10.11 CLOSING REMARKS

As the society grows and global cleavages increase, important buildings have become sitting targets of terrorist attacks. It is thus imperative that blast loads and the corresponding response of the building be studied. The current code, though exhaustive in nature, was written in 1969. International standards have considerably improved to include concepts like performance based analysis and progressive failures for such loads. The present discussion has been focused on the World Trade Centre collapse and progressive failure case study in the US, to give an overview of Blast Loads.

Effects of Tsunami have been covered only in terms of recommendations for Rehabilitation and structural measures that can be taken to mitigate the effects. But most of the literature refers to such recommendation without any specific details on how the particular solution can be achieved. Thus Mitigation aspects are mostly based on land use planning and local building elements detailing which might be able to save the structures even if they were damaged badly.