DUCTILITY AND PREVENTION OF STRUCTURAL FAILURE

by
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Abstract

This paper presents a review of the concept of ductility and its role in preventing failure of building structures under various types of loading. The scenarios under review here include structural distress due to vertical static load, differential support settlement, impact loading, machine vibration, blast loading and cyclonic storm. Moreover, the degradation of mechanical properties of concrete and steel due to fire is also shown. Assessment and illustrations from different severe loading conditions around the world are cited here from available literature. The structural damage and distress is attributed to a combination of lack of awareness and respect for existing rules in the design, construction and use of the structures, as well as absence of energy absorbing measures and adequate ductility. The paper then discusses various possible detailing/shock-absorbing options for these structures. Some relatively new options are presented in more detail here, because of their greater potential to improve the ductility of the structures. Finally, some relevant research works performed at the University of Asia Pacific (UAP) are presented.

Keywords

Ductility, Support settlement, Impact, Blast, Machine Vibration, Wave, Retrofit, Detailing, Nonlinear Dynamic Analysis.

1. INTRODUCTION:

While considering the strength of any element the applied load can be considered to be of a gradual or an instantaneous nature. The element may be subjected to cyclic or repetitive loading or it may involve impact or suddenly applied loads.

For example, impact loads produce high strain rates in the range of 10^0~10^2 per second and blast loads typically produce even higher strain rates in the range of 10^2~10^4 s^{-1}, while the more commonly applied quasi-static loads have a much smaller strain rate within the range of 10^{-6}~10^{-5} s^{-1}. This high loading rate would alter the dynamic mechanical properties of target structures and accordingly, the expected damage mechanisms for various structural elements. For reinforced concrete structures subjected to blast effects the strength of concrete and reinforcing steel bars can increase significantly due to strain rate effects. Fig. 1 shows the approximate ranges of the expected strain rates for different loading conditions.

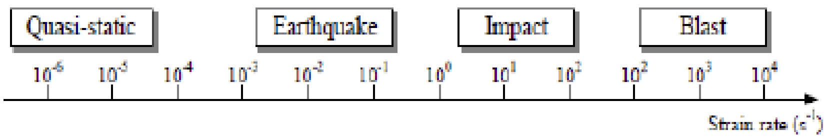

Fig. 1: Strain rates associated with different types of loading

1.1 Ductility

Ductility is the property of materials or structures to offer resistance in the inelastic domain of response. It includes the ability to sustain deformations in the inelastic range without significant loss of strength. This may translate to the capacity to absorb energy by hysteretic behavior during earthquakes or other time-varying loads. Increase in ductility of the structure helps the building to resist earthquake without enduring too much force.

The meaning of ductility for reinforced concrete structures can be summarized as follows:
* Re-distribution of internal forces among various structural members
* Reduction of internal forces due to restraints
* Indication of failure of structural distress

* Mobilization of capacity reserves for partial failure/damage
* Dissipation of energy for impact like earthquake, dynamic impact, or explosion

Some construction rules are implemented in many building codes in order to provide sufficient ductility of reinforced and pre-stressed concrete members.

2. STRUCTURAL DISTRESS UNDER VARIOUS LOADING CONDITIONS:

2.1 Quasi-static Loads

2.1.1 *Vertical Loads*

Most structures are designed primarily to support gravitational loads due to their self-weights and the weights they are supposed to carry to make them serviceable. Structural members like slabs, beams, columns and walls are to be designed to carry these loads from the super-structure to sub-structure, which in turn transfer the loads to the soil underneath. However, service requirements coupled with careless use often result in structural members being loaded beyond their load carrying capacity, and therefore over-stressed to distress or collapse. With increasing demands of population, and the ever-growing requirements of economy and business, this has unfortunately been a common cause of structural distress in Bangladesh.

Fig. 2 shows such scenarios of flexural/shear/axial cracks developed in structural members, while Fig. 3 demonstrate the ultimate collapse of overloaded buildings, often coupled with other irregularities like weak materials and poor construction.

Fig. 2: Various cracks developed in beams and columns due to overload

Fig. 3: Ultimate collapse of overloaded structure in (a) Bangladesh and (b) India

2.1.2 *Support Settlement*

In addition to overload, support settlement can be another common cause of structural distress. Here also, the demands of population and economy, coupled with casual and neglectful construction practices (e.g., filling up lands, ponds or lakes with soft soil, no/inaccurate soil testing or improvement practice) make it a potentially serious structural problem in Bangladesh.

Fig. 4 illustrates the problem of support settlement, and demonstrates that differential settlement can cause more structural distress than uniform settlement.

Fig. 4: (a) Sinking of entire structure due to support settlement

Fig. 4: (b) Building before support settlement, (c) Uniform settlement, (d) Differential settlement

The following are some of the most common signs of foundation problems due to support settlement.

* Sinking or Sloping floors [Fig. 5(a)]
* Cracks in brick and block [Fig. 5(b)]
* Foundation wall cracks [Fig. 5(c)]
* Wall rotation [Fig. 5(d)]
* Cracked or bowing walls [Fig. 5(e)]
* Cracked floors [Fig. 5(f)]
* Separation of windows from house [Fig. 5(g)]
* Walls separating from house [Fig. 5(h)]

Fig. 5: (a) Sinking or Sloping floors, (b) Cracks in brick and block, (c) Foundation wall cracks, (d) Wall rotation, (e) Cracked or Bowing Walls, (f) Cracked floors, (g) Separation of windows from house, (h) Walls separating from house

2.1.3 *Extreme Temperature (Fire)*

Exposure to fire has significant effects on all building materials. While this has recently become one of the more serious issues for structures in Bangladesh, never was the effect of fire more tragically exposed as after the terrorist attacks on September 11, 2001, where the structures were able to withstand the initial impact loads from aeroplanes, but failed subsequently when the steel columns yielded as steel yields as it is heated to high temperatures (Fig. 6).

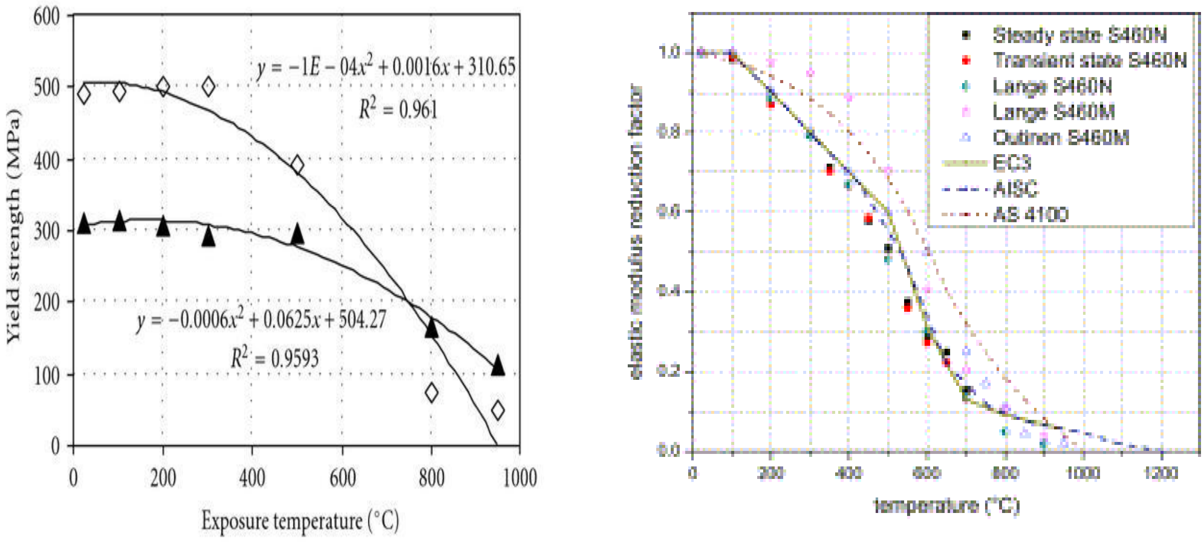

Fig. 6: Effect of temperature on (a) Yield strength, (b) Modulus of elasticity of steel

For concrete products, exposure to fire has, in many cases, been shown to have limited negative impact on the performance of the material. Concrete is regularly used to achieve fire-rating requirements of 1 to 4 hours, and has a good history of ease of repair after exposures to fire where the concrete has been heated to 260°C (500°F) or more.

Concrete temperatures up to 95°C (200°F) have little effect on the strength and other properties of concrete. Above this threshold cement paste shrinks due to dehydration and aggregates expand due to temperature rise. For normal-weight concrete the aggregate expansion exceeds the paste shrinkage resulting in an overall expansion of the concrete. In addition to the expansion, reductions in strength, Modulus of elasticity, and thermal conductivity occur, as well as increased rate of creep as temperature rises. In rare instances aggregates may exhibit abrupt volume changes at a particular temperature. Strength loss of concrete due to fire is illustrated in Fig. 7.

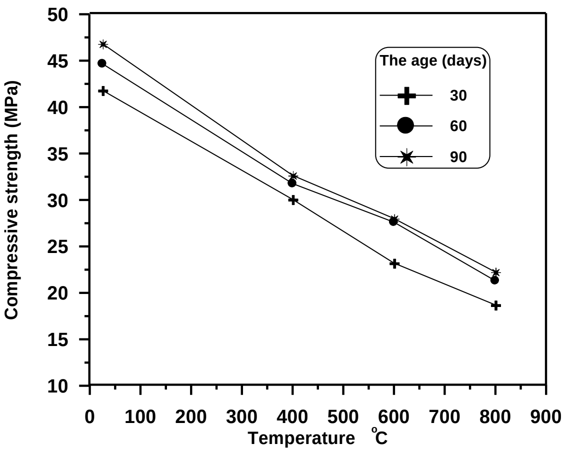

Fig. 7(a): The effect of fire flame on the compressive strength at 1-hour of exposure

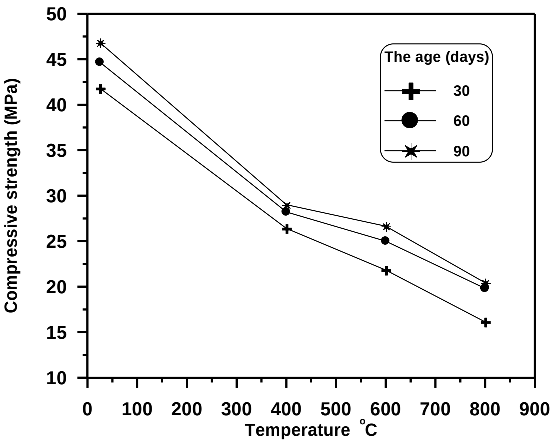

Fig. 7(b): The effect of fire flame on the compressive strength at 2-hour of exposure

Strength loss at high temperatures is due to the dehydration of the paste in the concrete matrix. As temperature rises, the dehydration of the paste will lead to a loss of essentially all of the concrete strength. In a fire event with fire exposure temperatures of 920°C (1700°F) the temperature within a concrete section at the depth of the clear cover protection (2 in) the actual concrete temperature at a 2-hour fire exposure may be below 260°C (500°F), this temperature being below the point at which steel reinforcement will begin to yield.

2.2 Impact Loads

The term impact refers to a dynamic effect of a load applied suddenly. If a load has kinetic energy and strikes instantaneously on a body, then the load is termed as impact load. The impact or sudden load condition may occur in tension, compression, torsion or bending or combinations of these. While considering the strength of any structural element it is almost invariably assumed that the load it will be subjected to is static or applied gradually. However, sometimes the loading case can be considered to be of an instantaneous nature. The element may be subjected to cyclic or repetitive loading or it may involve impact or suddenly applied loads.

Several common and important examples of impact loading are encountered regularly in our daily lives; including running, playing popular games like football, cricket, tennis, carom, pelting of stones or terrorist attacks including blast loads, and of course various types of vehicular collision. Impact loads result in shock waves propagating through the elements with possible serious consequences.

Extreme loading conditions like impact loads occur at a high rate of speed and transfer a large amount of energy into a structure over a short period time causing extreme local deformations and damage to structure. Structures can be exposed to extreme loads in their lifetimes.

2.2.1 *Progressive Failure of Slabs*

A building undergoes progressive collapse when a primary structural element fails, resulting in the failure of adjoining structural elements, which in turn causes further structural failure, similar to a house of cards.

Since the resulting damage in a progressive collapse is disproportionate to the original cause, the term disproportionate collapse is frequently used in engineering to describe this collapse type.

The first date-recorded instance of the term pancake collapse being published in lieu of 'progressive collapse' occurred in the August 10, 1980 edition of the New York Times. Fire Chief John Connelly of the 19th Battalion explained that the apartment building, which they responded to in the Bronx, had been weakened by fire to the point that all floors had begun to pancake down on one another. 'It was a pancake collapse' said Chief John Connelly of the 19th Battalion. 'The entire building was flaming and it went down to the ground.'

The sudden drop of the top slab causes a large impact load on the slab below, which it is unable to withstand and collapses as well. This creates a series of slab failures heaped on one another like a pack of cards (called a 'pancake' failure). Figs. 8(a)~(b) show pancake failures of overloaded slabs in USA and Bangladesh.

Fig. 8: Progressive Failure of slabs in (a) USA (1981), (b) Bangladesh (2007)

2.2.2 *Vehicular Impact on Bridge Railings*

However, Reinforced Concrete structures are rarely designed considering the effect on their behavior under impact load.

Vehicular collision with traffic barrier is a common case of impact. The primary function of a traffic barrier is to contain or safely redirect errant vehicles away from fixed features or to (occasionally) protect workers, pedestrians, or bicyclists from vehicular traffic. They can be installed as roadside or median barriers and are used to reduce the overall severity of collisions that occur when a vehicle leaves the traveled way. Barriers are designed so that such encounters might be less severe and not lead to secondary or tertiary collisions. However, when impacts occur, they are not guaranteed to redirect vehicles without injury to the occupants or additional collisions. Barrier performance is affected by the characteristics of the types of vehicles that collide with them. The safe design of traffic barrier is of paramount importance, since a vehicular crash involves the life and safety of several passengers, and in addition to the loss of life and property may seriously dent the morale of families and communities, if not the entire nation.

However, the performance of railings has not been satisfactory worldwide particularly when subjected to fast-moving vehicles veering off track. Fig. 9(a) shows a damaged railing and crash involving smaller vehicle, while Fig. 9(b) shows railing crash involving a larger vehicle.

Fig. 9: Railing crash involving (a) smaller vehicle, (b) larger vehicle

Fig. 10 shows the vehicle crash tests for RC railings, which has been established as a necessity by the American Federal Highway Administration (FHWA) and AASHTO, which however may be waived if an analytical evaluation shows the railing to be crash worthy.

Fig. 10: Arrangements for vehicular-impact test of RC railings

2.3 Machine Vibration

With the growing requirements of economy, building structures are now required to support heavy machineries, some of which may even vibrate on the foundations below. Added to that the demands of intermittent power supply, power generators are a common feature in most industries.

Here also, the mindless installation and use may render the structure exposed to the most damaging sequence of cyclic loads, as illustrated in Fig. 11, by the dynamic amplification of machine vibration, most prominently near the resonant frequency and even more so for less damped structures.

Moreover, the fatigue induced within the structural members due to repeated cycles of vibration may cause cracks within and make the structure even weaker to withstand the loads it is designed for.

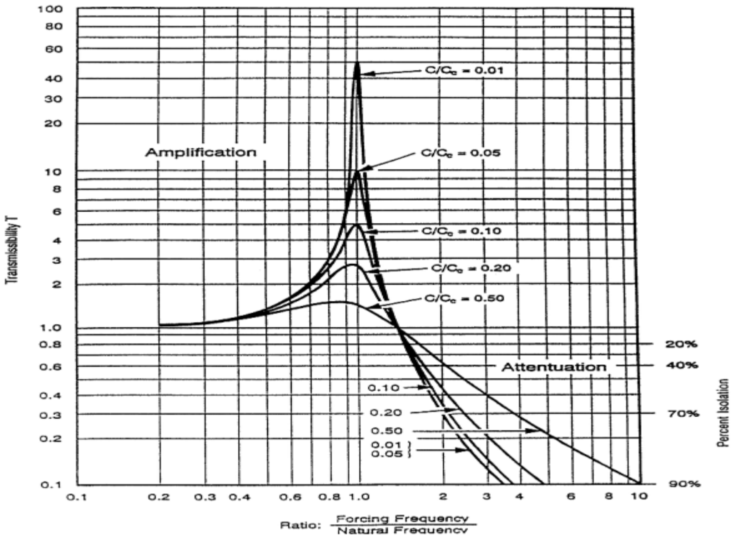

Fig. 11: Dynamic amplification of machine vibration

2.4 Blast Loading

Since the blast problem is relatively new; information about the development in this field is also not very widely known, some publications are considered classified secrets and therefore also not readily available.

In a developing country like Bangladesh, with much more common and pressing engineering challenges, the wealth of knowledge and available literature on this topic is rather thin. However, our country being within the global community, the problem of blast resistant design cannot be completely overlooked. Moreover, with the rise of extremist views and ideas within the region and increasing access to explosive materials, engineers realize the impact that even one single blast may have on a nation's history and its citizens' lifestyle, as demonstrated by the incident shown in Fig. 12 on the entire world's history.

Fig. 12: Blast load on World Trade Center (September 11, 2001)

Moreover, there is another possible positive application of blast loading in the controlled demolition of existing structures (Fig. 12). With the ever-changing urban infrastructure in this country and the predicament demonstrated by the demolition of even one single building, this may present a much better option for the building owners, construction workers and engineers.

Fig. 13: Controlled Demolition of structures

Blast loads create shock waves that propagate through the surrounding medium generating tremendous pressure on the objects they act upon. To provide some idea about the enormous pressures involved in blast loadings for engineers who often deal with floor loads in the order of 100 psf (less than 1 psi) and wind loads of about 30 psf (about 0.2 psi), Table 1 shows the range of pressures involved in blast loading and the resulting level of human fatality or injury. When the case of a car bomb explosion of 500 pounds TNT equivalence is considered, the Table 2 lists overpressures at gradually increasing stand-off distance. Fig. 14 shows the approximate variation of blast pressure with distance R, for explosives of different weights.

Table 1: Injuries for Effective Overpressure Duration of 100 Milliseconds

Injuries	Fatalities (for Effective Overpressure in psi)
Eardrum Rupture	5
Lung Damage a. Threshold b. Severe	 12 (Range: 8~15) 25 (Range: 20~37)
Lethal a. Threshold b. 50% c. 100%	 40 (Range: 30~50) 62 (Range: 50~75) 92 (Range: 75~115)

Table 2: Stand-off Distance and Overpressures (for 500 lb TNT Equivalence Explosion)

Stand-off Distance (ft)	Approximate Overpressure (psi)
50	10
100	7
200	3
300	1.4
500	0.8

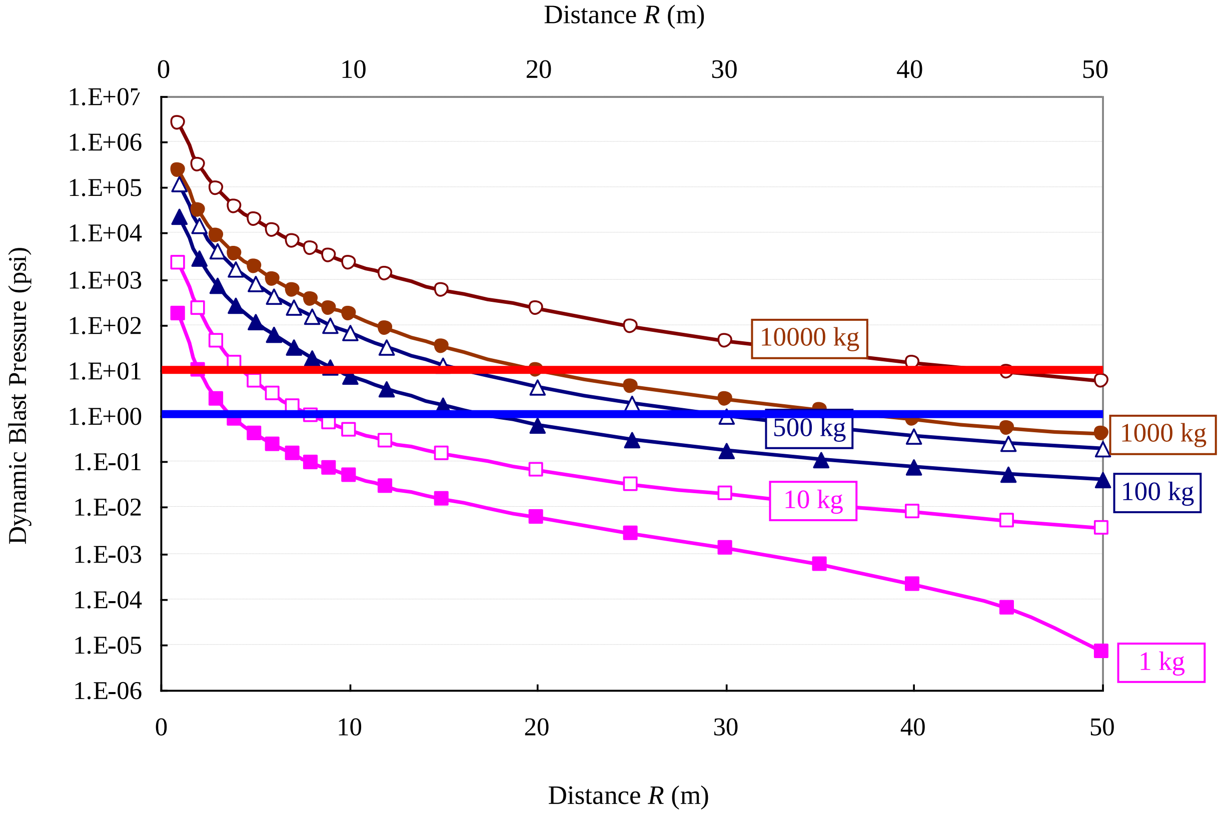

Fig. 14: Variation of blast pressure with distance, for explosives of different weights

2.5 Cyclone Loading

Bangladesh has always been a disaster prone country. Almost every year various kinds of disasters like excessive rainfall, flood, cyclone, drought affect the country. Among these disasters cyclone is a nightmare for the coastal districts. A large number of districts in the coastal areas are mostly affected by these cyclones. The Bay of Bengal is the breeding ground for tropical cyclones and Bangladesh is the worst victim in terms of fatalities and economic losses incurred. In the last 100 years 508 cyclones have originated in the Bay of Bengal, 17 percent of those have hit Bangladesh, amounting to a severe cyclone almost once every three years.

More than three million people live in high-risk areas along the 400-km coastline. The deadly cyclone of 1991 killed about 138,000 people, injured about the same number and left more than 300,000 homeless, bringing the count of those affected by the disaster to about 15.5 million, with an estimated damage of about $2 billion. Following this, the government and several international NGOs initiated a disaster preparedness and management program, including construction of cyclone shelters in vulnerable coastal areas. Today, a disaster warning system and evacuation procedure are in place, alongside more than 2500 multi-storied concrete cyclone shelters along the coastline. As a result, in the severe cyclone of 1997, the number killed and injured were 111 and about 10,000 respectively, demonstrating a great improvement over the 1991 figures. However, it had still left about one million people homeless. The figures from the more recent cyclone Sidr (2007) and Aila (2009) were even worse, both in terms of loss of lives, injuries as well as property damage.

Every cyclone leaves behind a huge damage, including lives, shelters and other properties. The majority of them have claimed more than five thousand lives each. These cyclones cannot be stopped but human lives and properties can be saved by taking appropriate precautions. Proper design guidelines need to be followed in building structures in the coastal region, but there is lack of proper design guide for building in the coastal regions of Bangladesh.

The high number of casualties in cyclones is due to the fact that they are always associated with storm surges. Storm surge height in excess of 6m is not uncommon in the coastal region. The elevation of land is only about 4 m at 2.5 km from the seashore where it is around 7 m even at 100 km inland from the seashore in western and central regions of the country. Moreover there are many low lands, flood plains, rivers and channels within this 100 km range. Thus the southern regions usually go under water during surge and face uncountable damage by destructive wind speed.

These cyclones cause damage to coastal buildings. In the coastal regions, usually, poor people cannot afford a concrete building, which may sustain in the high wind and wave forces. These buildings are usually made of low cost local materials without any engineering knowledge. Life of coastal people may change a lot if engineering knowledge can be used to improve the designs and materials. The country in general including the rich class, the structural designers and decision makers seem to be extremely insensitive and quite oblivious of their suffering. Technocrats do not adequately support housing project for low-income and flood vulnerable communities. The usual tendency is to apply the same model irrespective of context.

A comparatively new trend is the development of tourist facilities (including hotels, restaurants) along the coastlines. The threat of cyclone-induced loads is quite real on these multi-storied structures.

In addition to the cyclones, tsunamis present another threat looming large on the coastal structures. With the increased threat of earthquakes in Bangladesh, the risk of tsunamis poses a new threat that had never been encountered even in this disaster-prone country.

2.5.1 *Cyclones in Bangladesh*

A cyclone is one of a family of tropical storms (also called hurricanes, typhoons, or whirlwinds) that develop over warm tropical oceans and have sustained winds of 64 knots (74 mph or 120 kmph). Cyclones develop over warm oceans that are over 27ºC in temperature. Water evaporating from the sea acts as a kind of ‘fuel’, producing the energy of a cyclone. In one day, the energy released by a cyclone is at least 8,000 times more than the electrical power generated each day in the USA. Not only are the winds dangerous but they also blow on the water, creating the problem of storm surges and huge waves. Water can rise as high as thirty feet and floods can occur up to 30 miles inland.

The coastal zone of Bangladesh, an area covering 47,211 km² facing the Bay of Bengal or having proximity to the Bay, and the exclusive economic zone in the Bay, is generally perceived to be a zone of multiple vulnerabilities. Records of last 200 years show that at least 70 major cyclones hit the coastal belt of Bangladesh and during last 35 years nearly 900,000 people died due to catastrophic cyclones. It has also been found that number of occurrences of major cyclones has drastically increased in the recent decades. While the number of cyclones was 3 during the period of 1795~1845 and 1846~1896 respectively, the number increased to 13 during 1897~1947 and 51 during the period of 1848~1998.

Table 3: Categories of Cyclone

Category	Wind Speed (kmph)	Sustained Wind speed (kmph) (3 seconds gust)
1	120~155	145~185
2	156~180	186~225
3	181~210	226~265
4	211~250	266~315
5	Greater than 250	Greater than 315

Table 4: Typical Storm Surge Height for Cyclones in Bangladesh

Wind Velocity (Km/h)	Storm Surge Height (m)
85	1.5
115	2.5
135	3.0
165	3.5
195	4.8
225	6.0
235	6.5
260	7.8

Table 5: Past Devastating Cyclones in Bangladesh

Date	Year	Max. Wind Speed (Km/hr)	Storm Surge Ht. (m)	Deaths
09 Oct	1960	162	3	3,000
30 Oct	1960	210	4.5~6	5,149
09 May	1961	146	2.5~3	11,466
28 May	1963	203	4~5	11,520
11 May	1965	162	4	19,279
12 Nov	1970	223	6~10	5,00,000
25 May	1985	154	3~5	11,069
29 April	1991	225	6~8	1,38,000
15 Nov	2007	240	5~6	3,406
25 May	2009	120	2~3	330

Fig. 15: Images of cyclones in (a) Bhola (1970), (b) Chittagong (1991)

2.5.2 *Loads due to Surge (BNBC 1993)*

For the determination of surge loads on a structural member, consideration shall be given to both hydrostatic and hydrodynamic effects. Required loading shall be determined in accordance with the established principles of mechanics based on site-specific criteria and in compliance with the following provisions of this section. For essential facilities like cyclone and flood shelters and for hazardous facilities, values of maximum flood elevation, surge height, wind velocities etc., required for the determination of flood and surge load, shall be taken corresponding to 100-year return period. For structures other than essential and hazardous facilities, these values shall be based on 50-year return period.

For structures sited at coastal areas, the hydrostatic and hydrodynamic loads shall be determined as follows

<u>*Hydrostatic Loads*</u>

The hydrostatic loads on structural elements and foundations shall be determined based on the maximum static height of water, H_m produced by floods or surges as given by

$$H_m = \text{Max}(h_s, h_f) \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots(1)$$

where $h_f = y_T - y_g$

h_s = Maximum surge height as specified in (i) below

y_T = Elevation of the extreme surface water level corresponding to a T-year return period specified in (ii) below, meters

y_g = Elevation of ground level at site, meters.

(i) Maximum Surge Height, h_s: The maximum surge height, h_s, associated with cyclones, shall be that corresponding to a 50-year or a 100-year return period as may be applicable, based on site specific analysis. In the absence of a more rigorous site-specific analysis, the following relation may be used:

$$h_s = h_T - (x - 1)\, k \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots(2)$$

where h_T = Design surge height corresponding to a return period of T-years at sea coast, in meters, given in Table 6 (i.e., Table 6.2.28 of BNBC'93).

x = Distance (km) of the structure site measured from the spring tide high-water limit on the sea coast, in km; $x = 1$, if $x < 1$.

k = Rate of decrease in surge height in m/km; the value of k may be taken as 1/2 for Chittagong-Cox's Bazar-Teknaf coast and as 1/3 for other coastal areas

Table 6: Design Surge Heights at the Sea Coast, h_T

Coastal Region	Surge Height at the Sea Coast, h_T (m)	
	T = 50-year	T = 100-year
Teknaf to Cox's Bazar	4.5	5.8
Chakaria to Anwara, and Maheshkhali-Kutubdia Islands	7.1	8.6
Chittagong to Noakhali	7.9	9.6
Sandwip, Hatiya and all islands in this region	7.9	9.6
Bhola to Barguna	6.2	7.7
Sarankhola to Shyamnagar	5.3	6.4

(ii) Extreme Surface Water Level, y_T: The elevation of the extreme surface water level, y_T for a site during monsoon, which may not be associated with a cyclonic storm surge, shall be that obtained from a site-specific analysis corresponding to a 50-year or a 100-year return period. Values of y_T are also given in Table 6.2.29 of BNBC'93 for selected coastal locations to be used in the absence of any site-specific data.

<u>*Hydrodynamic Loads*</u>

The hydrodynamic load applied on a structural element due to wind-induced local waves of water, shall be determined by a rational analysis using an established method and based on site-specific data. In the absence of a site-specific data the amplitude of the local wave, to be used in the rational analysis, shall be taken as $h_w = h_s/4 \geq 1$-m. Such forces shall be calculated based on 50-year or 100-year return period of flood or surge. The corresponding wind velocities shall be 260 km/h or 289 km/h respectively.

3. DUCTILITY PROVISIONS AND STRUCTURAL RETROFIT:

3.1 Ductility Provisions in Structural Design

3.1.1 *Provisions for Quasi-Static Load*

Structural engineers are quite aware of the fact that depending on the nature of collapse, the failure of a RC flexural member (e.g., beams, slabs, shallow footings) can be of two types. The first is *Tension Failure*; i.e., when the steel reinforcements would yield ($f_s = f_y$) as concrete reaches its ultimate crushing strength f_c'. However, if the steel ratio is too high, the reinforcements may not yield when the concrete reaches its capacity. This failure mode, i.e., the *Compression Failure* occurs when the compression strain in concrete reaches its ultimate strain before the yielding of steel ($f_s < f_y$).

Between the two modes, compression failure occurs explosively and without any warning of distress. For this reason, it is good practice to keep the amount of reinforcement sufficiently small to ensure that, should the member be overstressed, it will give adequate warning before failing in a gradual manner by yielding of steel (accompanied by deflections and widening of concrete cracks) rather than by crushing of concrete. This can be done by keeping the reinforcement ratio below a certain limiting value. This value, the so-called *Balanced Steel Ratio* (ρ_b), represents the amount of reinforcement necessary to make the beam fail by crushing of concrete at the same load that causes the yielding of steel. In practice, however, the maximum steel ratio (ρ_{max}) is kept smaller than ρ_b [ACI recommends $\rho_{max} = 0.75\rho_b$]. On the other hand, the steel ratio should not also be so small that the effect of reinforcement is negligible and the flexural member would collapse as soon the concrete reaches its tensile strength.

Both the maximum and minimum steel ratio is implemented with the view to ensure ductility; i.e., so that the flexural member fails in a ductile manner.

The same objective guides the choice of transverse reinforcements in axially loaded members (e.g., columns, piles), which ensures that the longitudinal reinforcements do not fail before reaching their ultimate capacity. Figs. 16 and 17 show the structural arrangement and behavior of a tied column compared to a spirally reinforced column, the concrete spalling load of both columns being equal. However, it follows that the former fails in a sudden and brittle manner and the latter in a more gradual manner, which explains the relatively greater value of ϕ for spirally reinforced columns (= 0.75 compared to 0.70 for tied columns).

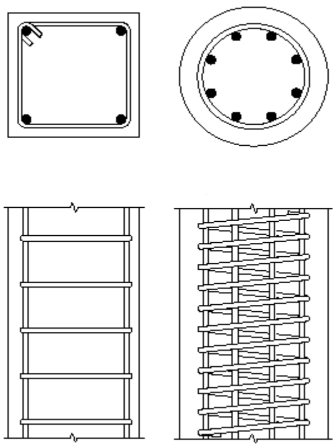

Fig. 16: Columns with Lateral Ties and Spiral Reinforcements

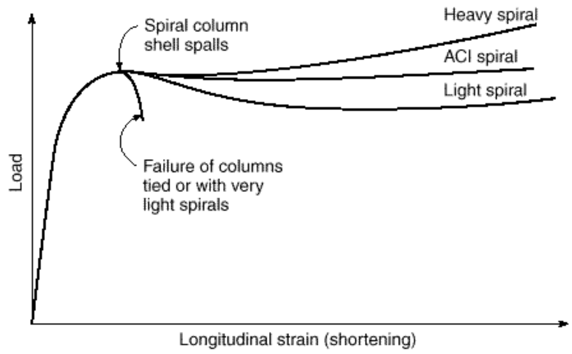

Fig. 17: Behavior of tied and spirally reinforced columns (Nilson)

3.1.2 *Provisions for Impact Load*

Impact force from a falling weight (e.g., a falling slab) can become very large, especially due to fall on bare concrete. In order to avoid such high forces, protection galleries are covered by gravel or some other damping material. An increased cushion depth reduces the impact forces further. However, additional factors influence the forces, such as the gravel’s degree of compaction or its behavior when rupture occurs.

Fig. 18 shows the arrangements of free fall tests on concrete slabs without and with a gravel cushion.

Fig. 18: Arrangements of free fall tests on concrete slabs without and with a gravel cushion

3.1.3 *Provisions for Machine Vibration*

One important aspect of machine vibrations that the users can often forget is the fact that machines should either be placed on rigid and well-designed foundations or if it is necessary to put them on structural slabs, they should be supported on shock-absorbing springs, similar to the arrangements shown in Fig. 19.

Fig. 19: Machines supported on shock-absorbing springs

Fig. 20 shows the time-dependent structural response of the supporting machine with and without the shock absorbing spring underneath. The remarkable reduction of the structural vibration results from a combination of reduction of stiffness due to spring flexibility (that reduces the natural frequency of the system to avoid the resonance) and increased damping, which further reduces the resonant amplitude.

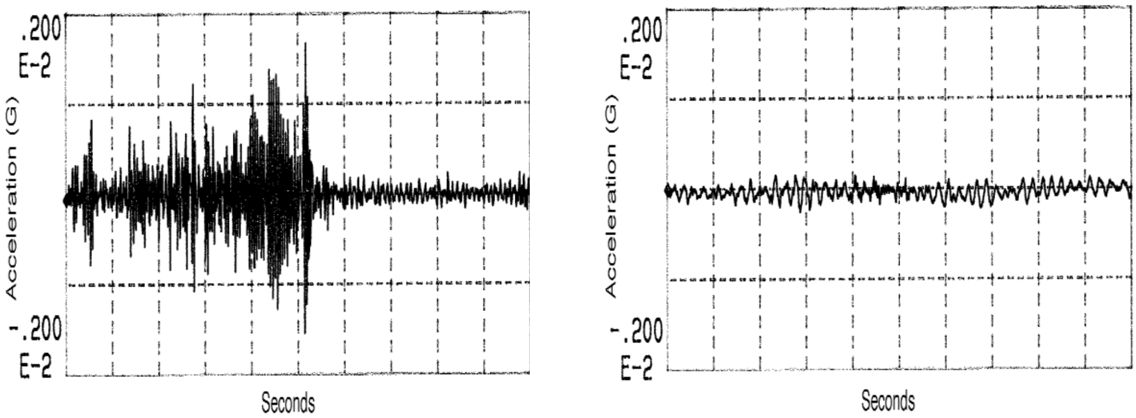

Fig. 20: Response time series of the structure supporting the machine (a) Without, and (b) With shock absorbing spring underneath

3.1.4 *Provisions for Cyclone Load*

Various studies show that mangrove forest and other coastal vegetation of certain density can reduce wave height considerably and protect the coast from erosion, as well as effectively prevent coastal sand dune movement during strong winds. Healthy coastal forests such as mangroves and salt marshes can serve as a coastal defense system where they grow in equilibrium with erosion and accretion processes generated by waves, winds and other natural actions. The coastal areas around the Bay of Bengal are vulnerable to strong winds, storm surges, tectonic movement, over-sedimentation, rapid coastal erosion, fluctuating water and soil salinity and long periods of constant flooding.

Based on these field studies, the wave and current characteristics of propagation through the mangrove forest area are as follows:

* The wave height of the swell increases with increasing tidal level, and decreases with increasing proximity to the coast, which suggests wave energy loss caused by bottom friction and resistance to flow by the mangrove vegetation.
* Wave size decreases considerably through denser mangrove areas; therefore, in well-grown and healthy mangrove areas, the effects on wave reduction do not decrease with increasing water depth, which has important practical implications.
* According to the research, the effectiveness of mangroves with *Kandelia candel* of sufficient height (three to four years old) in reducing wave height per 100 meters was as high as 20 percent and increased to 95 percent when the trees were six years old. At this age, 1-meter wave height on the open coast will be reduced to 0.05 meter at the coast compared to 0.75 meter without mangroves. Vegetation height and density and the width of the area to be planted are important factors in reducing wave height and protecting the coast from erosion. The effect of wave reduction was considerable even when water depth increased due to the high density of vegetation.

Fig. 21 is a schematic diagram of wave attenuation model by mangrove forest, while Figs. 20(a) and 20(b) show the spatial variation of tsunami and cyclone water depth and current velocity due to vegetation as well as higher soils or land.

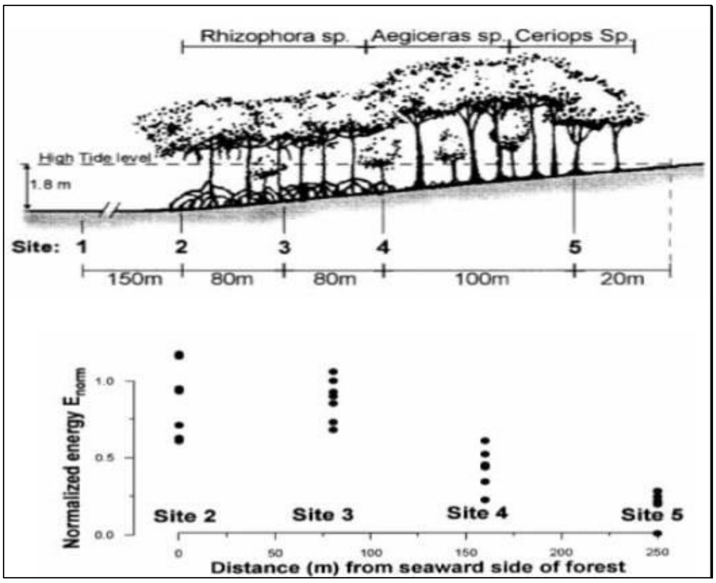

Fig. 21: Wave attenuation by mangrove forest

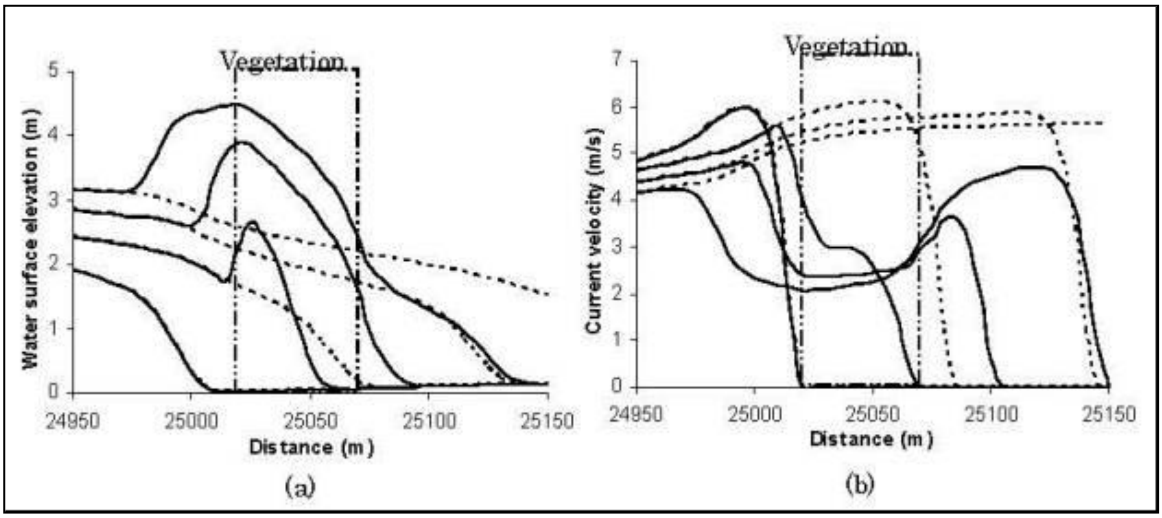

Fig. 22: Spatial variation of (a) water depth, and (b) current velocity for tsunami periods of 10 min

Fig. 23 shows the natural protective functions of coastal forest and trees, starting with water-edge vegetation on intertidal deltas, rising to hydric species on higher soils or land. Figs. 23(a) and 23(b) show the important role of coastal forest and vegetation to diminish the wave height of a 6 m tsunami to 1.6m, and consequently saving coastal structures at West Java.

Fig. 23: Coastal forest and vegetation (a) diminished 6-m tsunami wave height to 1.6m, (b) prevented destruction of houses at West Java

3.1.5 *Blast Resistant Design*

In relation to an external threat, the priority of blast-resistant design should be to create as much stand-off distance between an external bomb and the building as possible. On congested city centers there may be little or no scope for repositioning the building, but what small stand-off there is should be secured where possible. This can be achieved by strategic location of obstructions such as bollards, trees and street furniture. Fig. 24 shows a possible external and internal layout of blast-safe planning.

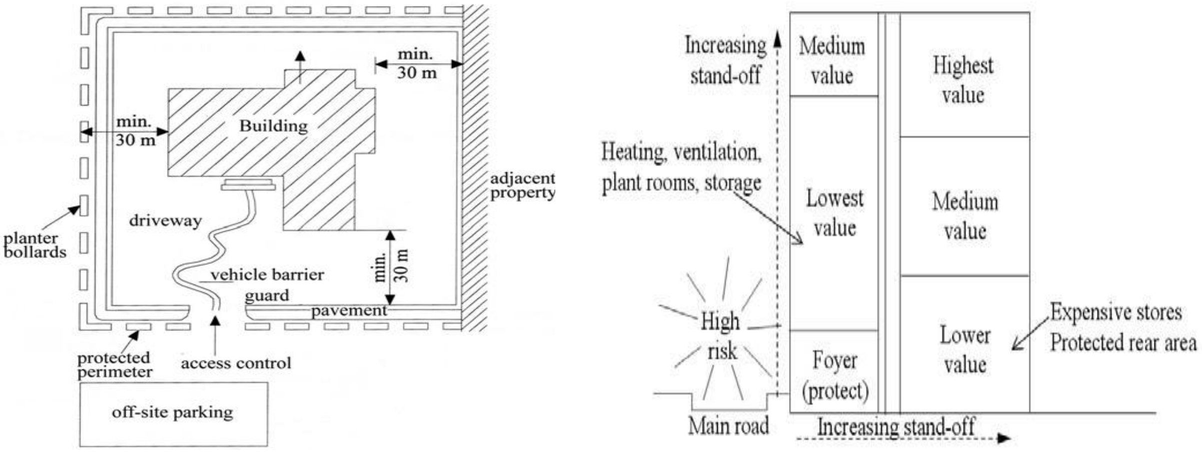

Fig. 24: (a) Schematic layout of site for protection against bombs, (b) Internal planning of a building

It is often stated that blast damage would be reduced if 'seismic-like' construction standards were adhered to. It is true that seismic building design details enhance the ductility of structures and thereby increase their capacity to sustain plastic hinges and withstand large rotations. Yet, it is important to understand that the nature of the blast loading and the response of structure to it are different in many ways from a seismic event.

Frame buildings designed to resist gravity, wind loads and earthquake loads in the normal way have frequently been found to be deficient in two respects. When subjected to blast loading; the failure of beam-to-column connections and the inability of the structure to tolerate load reversal. Beam-to-column connections can be subjected to very high forces as the result of an explosion. In the connections, normal details for static loading have been found to be inadequate for blast loading.

Fig. 25 shows the side-plate connection details. The main features to note in the reinforced concrete connection are the use of extra links and the location of the starter bars in the connection. These enhancements are intended to reduce the risk of collapse or the connection be damaged, possibly as a result of a load reversal on the beam. It is vital that in critical areas, full moment-resisting connections are made in order to ensure the load carrying capacity of structural members after an explosion. Two types of wrapping can be applied to provide this; i.e., wrapping with steel belts or wrapping with carbon fiber-reinforced polymers (Fig. 26).

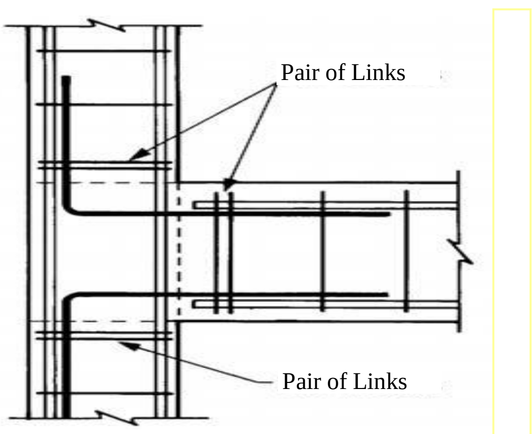

Fig. 25: Enhanced Beam-Column connection details for steelwork and Reinforced Concrete

Fig. 26: CFRP wrapped column

3.2 Methods of Structural Retrofit

3.2.1 *Jacketing*

Jacketing consists of encasing the existing element by an additional structural component. The RC jacketing method, unlike other techniques, leads to a uniformly distributed increase in strength and stiffness of columns. The durability of the original column is also improved, in contrast to the corrosion and fire protection needs of other techniques where steel is exposed or where epoxy resins are used. Finally, this procedure does not require specialized workmanship. All those reasons make RC jacketing an extremely valuable choice in structural rehabilitation.

The main purposes of jacketing are to increase

* Concrete confinement by transverse fiber/reinforcement, especially for circular columns
* Shear strength by transverse fiber/reinforcement
* Flexural strength by longitudinal fiber/reinforcement if well anchored at critical sections.

For jacketing common retrofit techniques include concrete, FRP and steel jackets. Concrete jackets are constructed by enlarging the existing cross section with a new layer of concrete and reinforcement. This reinforcement is traditionally provided by hoop or spiral rebar, or welded wire fabric. FRP reinforcement is typically applied two ways: prefabricated jackets or wraps. Both methods have been experimentally researched. Steel jackets are constructed by placing a steel tube with a slightly larger diameter around the member to be retrofitted.

<u>*Concrete Jacketing*</u>

Addition of a concrete jacket is used to enhance flexural strength, ductility and shear strength of columns. This technique is more commonly used for building columns but has been applied to some bridge members in Japan. The enhanced confinement is achieved with the use of ties or spirals at a small pitch or transverse reinforcement spacing (Priestley et al. 1996). Concrete jackets can be used to retrofit beams as well as columns. Additional materials can be used to reinforce the retrofit, as long as confinement is enhanced.

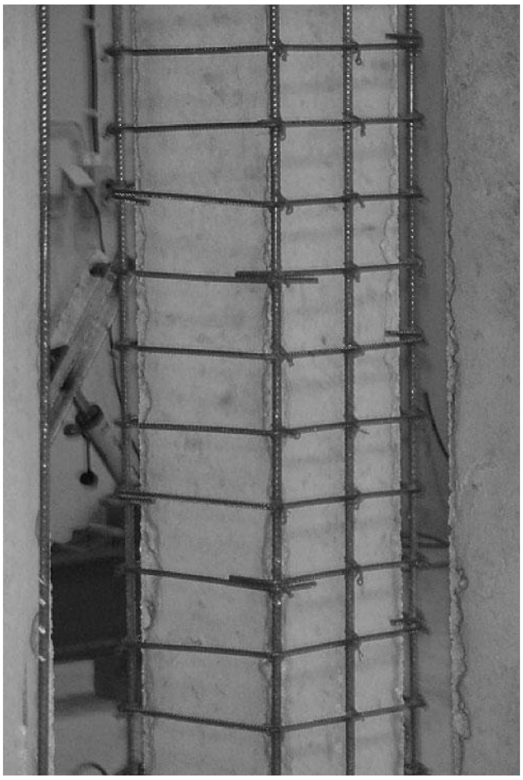

Fig. 27: Concrete Jacketing of RC columns

Steel Jacketing
Steel jackets prevent concrete from expanding laterally as a result of high axial compression strains. The steel jacket is equivalent to continuous hoop reinforcement and can be used for circular columns or rectangular columns with slight modifications, as shown in Fig. 28. Steel jacketing of rectangular columns is not recommended because while shear strength is enhanced, flexural ductility is only provided at the corners. Priestley et al. (1996) recommends that an elliptical steel jacket with concrete infill should be provided for rectangular members to fully confine all the concrete. ACI design equations are overly conservative and new design equations are presented for circular or rectangular columns in need of shear enhancement.

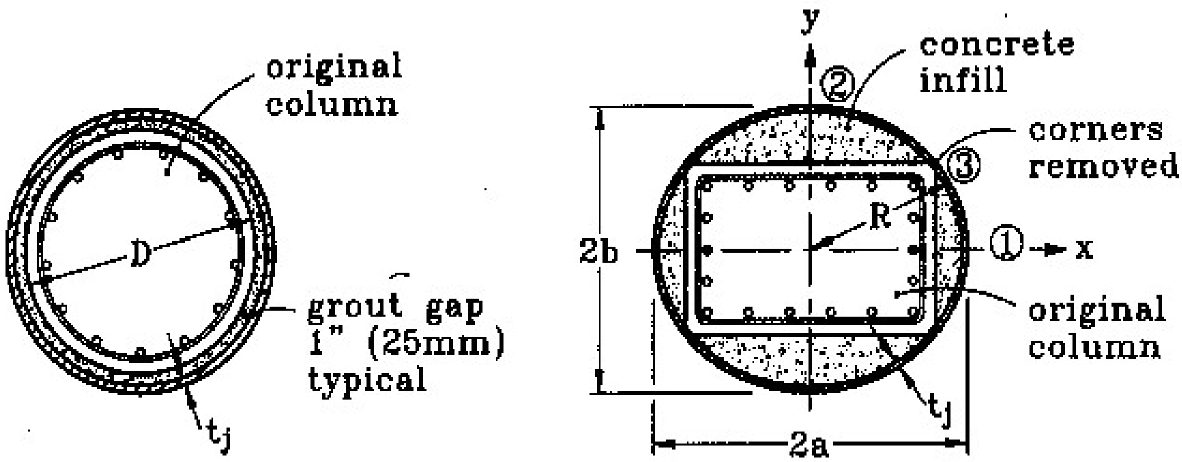

Fig. 28: Steel jacketed columns (a) circular, (b) rectangular with elliptical jacket [Priestley et al. (1996)]

Fig. 29: FRP jacketed (a) Circular Columns, (b) Square Columns (Benzaid et al. 2008)

FRP Jacketing
Fiber reinforced polymer (FRP) confinement can be provided using several composite materials including fiberglass, carbon fiber and Kevlar bonded to the confined concrete surface using epoxy (Priestley et al. 1996). Weight and cross-section of the retrofitted member are not related significantly to FRP jackets, which are most applicable for circular columns, as stress concentrations can develop in the FRP wrap around the corners of square or rectangular cross-sections. They can be used for rectangular or other shaped members, if shape

modification is performed to prevent stress concentrations from developing. Two types of FRP retrofits, wraps and prefabricated composite jackets, are typically used.

In FRP jacketing of the columns attention should be paid to the following aspects

* repair method of the original column – removing the concrete from the deteriorated zone by hand chipping, jack-hammering or electric hammering should be followed by sand-blasting or water demolition techniques.
* interface surface preparation – in case of an undamaged and sound element, there is no need to improve the roughness of the interface surface, except for the situation of short RC columns, where sand blasting or water demolition should be used.
* use of a bonding agent – a two-component epoxy resin is most commonly used. However, an effective method to increase the surface roughness, such as sand-blasting, is enough to enhance the interface strength, when justified. In this latter situation, the subsequent application of an epoxy resin can even produce the opposite result and therefore should not be used;
* application of steel connectors – this should be considered only in the case of short RC columns to improve the level of strength and stiffness under cyclic loading.
* temporary shoring – the implications of shoring the original column should be considered in such away that the RC jacket will resist part of the total load and not only part of the load increments.
* anchoring of the added longitudinal reinforcement – holes have to be drilled on the footings and appropriately cleaned. The use of a vacuum cleaner is highly recommended. The steel bars can be efficiently anchored to the footing with a two-component epoxy resin.
* continuity between floors of the added longitudinal reinforcement – holes must be drilled in the slab to allow steel bars to pass through. However, if the only objective is to increase the column shear strength and ductility, continuity is not needed and gaps should be provided instead.
* position of the steel bars of the longitudinal reinforcement – these should be uniformly spread. If this is not possible, attention must be paid to avoid excessive bundling at the corners.
* added stirrups – half of the spacing of the original transverse reinforcement is recommended for the added stirrups to obtain a monolithic behavior under cyclic loading. These should also be placed out of phase.
* added concrete – a non-shrinkage concrete should be adopted with characteristics of a self-compacting, high-strength and high-durability concrete.

3.2.2 *Confinement*

In a typical reinforced concrete member, the cover concrete will spall, or crack and separate, under loading before the center, or core, concrete. Transverse reinforcement provides confinement of the core concrete which significantly impacts the behavior of the concrete. Effectively confined concrete displays increased strength and ductility over unconfined concrete. Confined concrete also has the ability to carry larger stresses and strains at ultimate strength. This is evident in the stress-strain model proposed by Saatcioglu and Razvi (1992) shown in Fig. 30.

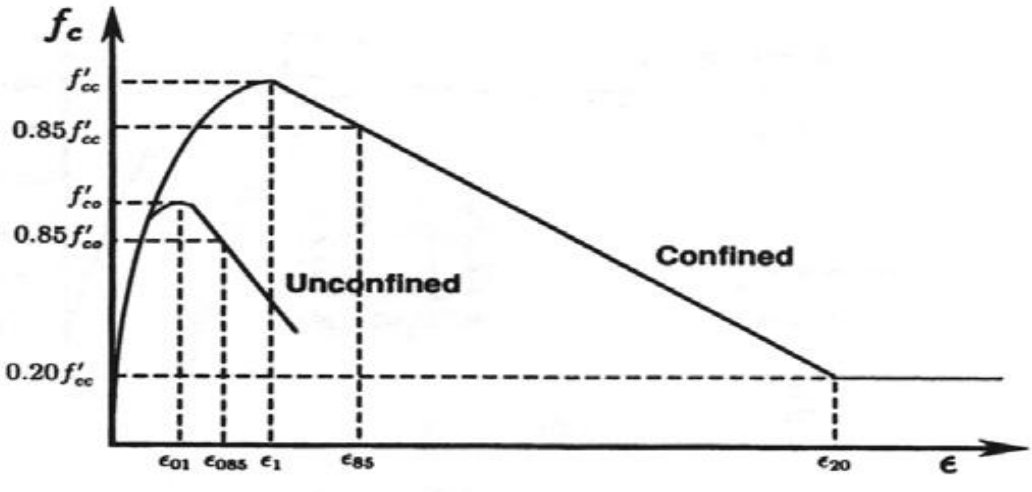

Fig. 30: Proposed Stress-Strain Relationship of concrete (Saatcioglu & Razvi 1992)

Behavior of confined concrete is dependent upon a variety of factors. Quantity, yield strength, configuration, and spacing of transverse reinforcement; compressive strength of concrete; and resulting arrangement of longitudinal steel are all parameters studied in the past. In addition, strain rate and cross-sectional geometry (Saatcioglu & Razvi 1992) are two other factors that have been researched. Concrete confinement models typically consist of two parts, the ascending branch and descending branch. The ascending branch is the section of the graph that leads up to the ultimate strength of the system and the descending branch follows after ultimate strength is reached. Using this terminology, confining concrete increases the height of the ascending branch and extends or lengthens the descending branch. Branch characteristics differ for fiber reinforced polymer and concrete filled steel tubes because of the nature of the applied confinement. Confinement can be provided in a variety of ways. Lateral reinforcement, typically rebar hoops or spirals have historically been used for confinement. Additionally, lateral reinforcement is provided to keep longitudinal reinforcement in place during casting and to prevent compressive reinforcement from buckling. Ideally, concrete structures in seismic regions are designed to meet more stringent lateral reinforcement requirements to provide the previously mentioned benefits.

Confinement Mechanism

Active confinement continually provides confining pressure, as is the case with fluid pressure. Passive confinement, such as that provided by spiral reinforcement, is not a constant pressure and is dependent on the lateral expansion of concrete from an axial load and the corresponding response of the confining steel. The strength of actively confined concrete is approximately the same as concrete with passive confinement provided by closely spaced spiral reinforcement. Small axial loads produce very small stresses in passively confined concrete, e.g. concrete confined by spiral reinforcement, and the concrete essentially is unconfined. As the axial stress increases, the concrete expands outwardly, due to progressive internal fracture, and bears against the transverse reinforcement which applies a confining response to the concrete (Kent & Park 1971), as shown in Fig. 31. Confinement arching is shown in Fig. 32.

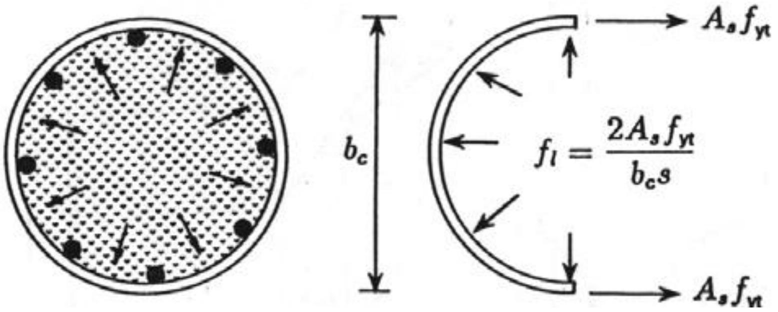

Fig. 31: Lateral Pressure in Circular Columns: (a) Uniform buildup of pressure and (b) Computation of Lateral Pressure from Hoop Tension (Saatcioglu & Razvi 1992)

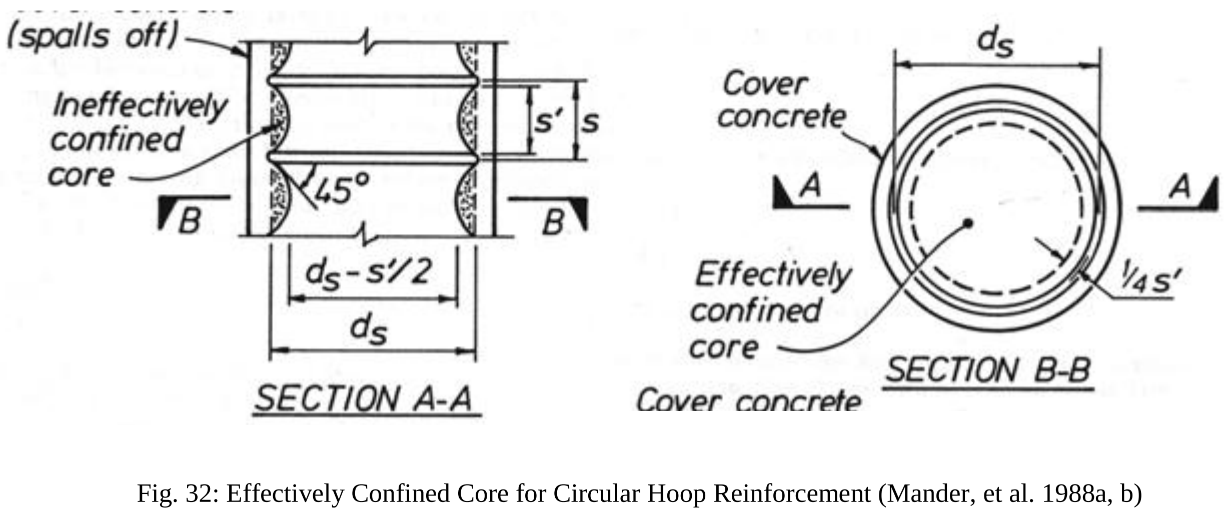

Fig. 32: Effectively Confined Core for Circular Hoop Reinforcement (Mander, et al. 1988a, b)

4. RELEVANT RESEARCH AT UNIVERSITY OF ASIA PACIFIC (UAP):

A number of research works have been performed at UAP on the behavior of RC to blast, wave and impact loading. A major portion of the work has been focused on the nonlinear dynamic behavior of the structure and on how to improve that. A brief description of the relevant numerical and experimental works is provided here.

4.1 Numerical Study on Design of Blast Resistant Buildings (Islam 2012)

Fig. 33 shows the model 6-, 12- and 24-storied buildings considered for structural analysis. The analyses are simplified significantly by modeling them as SDOF systems, the properties of which are derived from the structural column dimensions and slab loads.

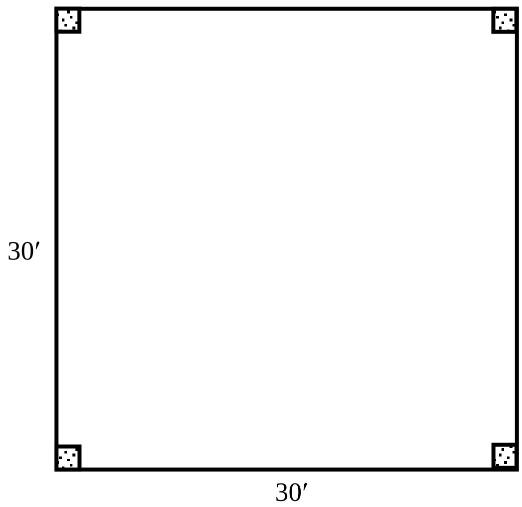

Fig. 33: Plan of 6-, 12 and 24-storied buildings

The columns sections designed for the different buildings are shown in Fig. 34. Using basic concepts of RC column design, column size of (12.5″ × 12.5″) is determined with 8 # 7 bars for the 6-storied building. Similarly (18″ × 18″) column with 12 # 8 bars is chosen for 12-storied building, while (25″ × 25″) column with 20 # 9 bars is chosen for 24-storied building.

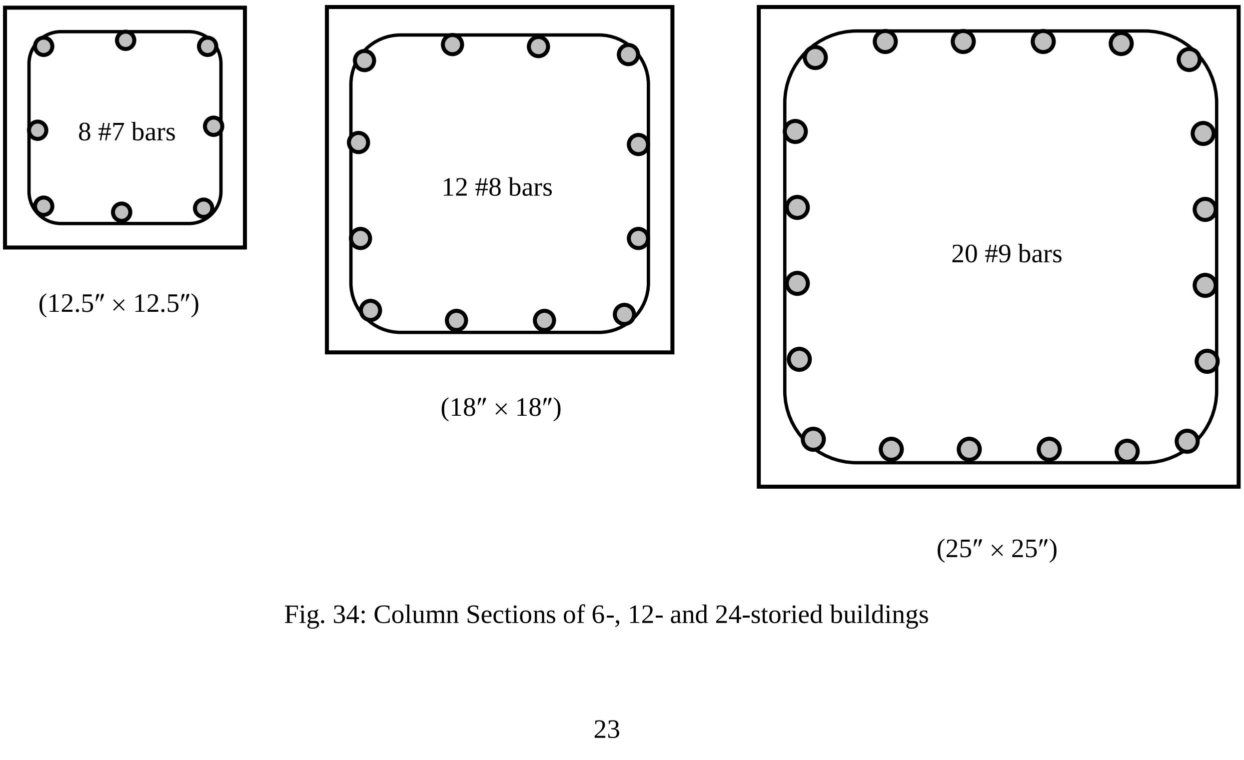

Fig. 34: Column Sections of 6-, 12- and 24-storied buildings

The actual structure can be replaced by an equivalent system of one concentrated mass and one weightless spring representing the resistance of the structure against deformation. The structural mass, *m*, is under the effect of an external force, *F*(*t*), and the structural resistance, *R*, is expressed in terms of the vertical displacement, *y*, and the spring constant, *k*. Such an idealized system is illustrated in Fig. 35.

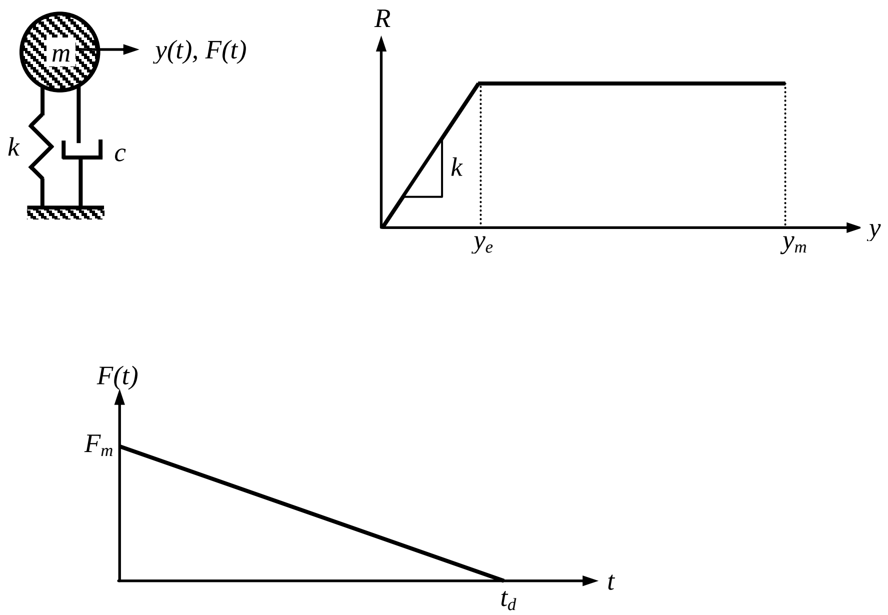

Fig. 35: (a) Damped SDOF system with elastic fully-plastic *k*, (b) Blast Loading

4.1.1 *Results from Numerical Analysis*

A wide range of damping ratio DR (= 0, 0.05, 0.10, 0.20), TR (0~10) and RR (0.1~2.0) are chosen to develop the 'design graphs' for the 'Ductility Demand' of the structure (i.e., maximum structural displacement compared to its 'elastic' deformation), as shown in Fig. 36.

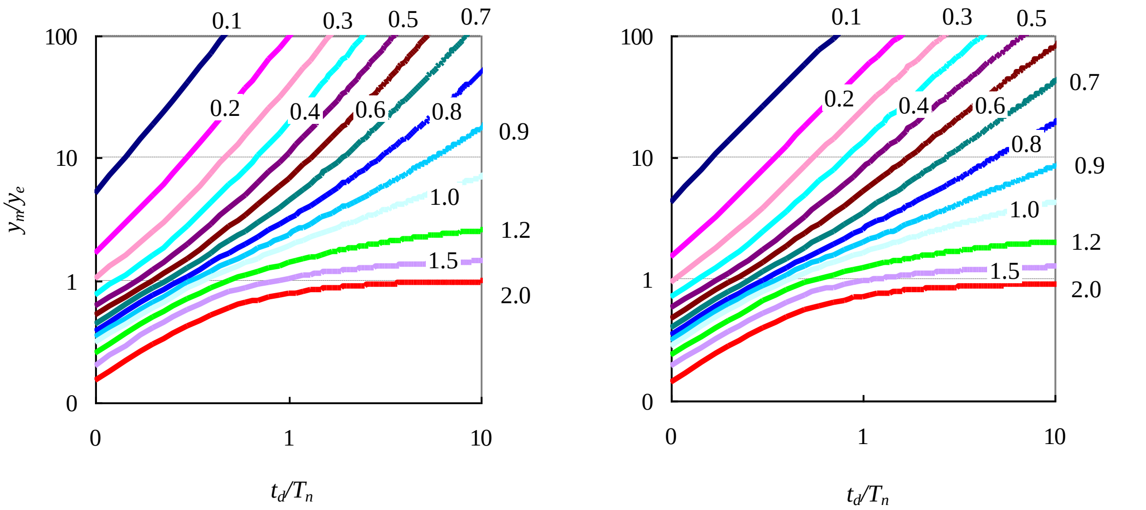

Fig. 36: Response of SDOF System to Blast Load for R_u/F_m = 0.10~2.0 and Damping Ratio (a) 0%, (b) 5%

The results from numerical analyses are summarized in Tables 7~9. The critical points of comparison are the values of Ductility Ratio (y_u/y_e) and Ductility Demand (y_m/y_e) for different explosive weights, duration ratio (t_d/T_n) and stand-off distance (R). While a distance of 3-m is found to be safe for a 100 kg explosive weight, it requires a 10-m stand-off distance for 1000 kg explosive, and at least a 30-m distance for 10000 kg explosive. The effect of seismic detailing is found to be important, though not as much as the importance of R and the weight of explosive.

Table 7: SDOF Properties for Different Structures

Column	k (k/ft)	y_e (ft)	y_u (ft)	R_u (k)	m (k-s^2/ft)	T_n (s)	y_u/y_e
6-00N	1.44E+03	1.06E-02	0.43	15.2	29.35	0.90	40.3
6-00M	1.27E+03	9.45E-03	3.83	12.0	29.35	0.96	406
6-100	1.33E+03	1.30E-02	6.14	17.3	29.35	0.93	472
6-1000	1.11E+03	1.69E-02	6.14	18.7	29.35	1.02	364
12-00N	2.18E+03	1.28E-02	0.21	28.0	58.70	1.03	16.6
12-00M	4.43E+03	4.89E-03	1.10	21.7	58.70	0.72	225
12-100	4.69E+03	6.61E-03	3.33	31.0	58.70	0.70	503
12-1000	2.43E+03	1.47E-02	3.24	35.7	58.70	0.98	221
24-00N	3.00E+03	1.44E-02	0.15	43.3	117.39	1.24	10.4
24-00M	6.00E+03	5.83E-03	0.73	35.0	117.39	0.88	124
24-100	5.95E+03	8.78E-03	2.10	52.2	117.39	0.88	239
24-1000	3.15E+03	1.87E-02	2.10	59.0	117.39	1.21	112

For Column 6-00N or 6-00M
6 ⇒ 6-storied Structure
00 ⇒ Strain Rate = 0/sec
N ⇒ No Seismic Detailing
M ⇒ Major Seismic Detailing

For Column 6-100 and 6-1000
100 ⇒ Strain Rate = 100/sec
1000 ⇒ Strain Rate = 1000/sec

Both use Major Seismic Detailing

Table 8: Maximum Blast Force on Different Structures

Distance R (m)	Maximum Dynamic Force, F (k)		
	W = 100 kg	W = 1000 kg	W = 10000 kg
3	3.18E+04	3.80E+05	3.88E+06
10	3.33E+02	7.00E+03	9.65E+04
30	7.73E+00	9.54E+01	1.91E+03

Table 9: Ductility Demand (y_m/y_e) for Different Loading Conditions

W (kg)	t_d/T_n	6-Storied				12-Storied				24-Storied		
		R = 3m	R = 10m	R = 30m		R = 3m	R = 10m	R = 30m		R = 3m	R = 10m	R = 30m
100	0.0125	356	0.68	0.016		153	0.34	0.008		90	0.23	0.005
	0.0250	847	1.55	0.033		386	0.73	0.017		239	0.49	0.011
	0.0500	1859	4.57	0.069		879	1.62	0.035		560	1.00	0.023
1000	0.0125	5242	51	0.194		2548	18	0.097		1653	10	0.064
	0.0250	11423	142	0.416		5620	56	0.208		3686	31	0.138
	0.0500	23818	347	0.857		11795	148	0.428		7781	87	0.285
10000	0.0125	55190	1246	6.91		27480	579	2.32		18266	363	1.34
	0.0250	118559	2802	22.97		59149	1343	7.71		39387	863	4.03
	0.0500	245327	5943	65.90		122519	2899	24.08		81661	1893	12.85

4.2 Dynamic Response of Coastal Structures to Ocean Wave Loading (Sharif 2012)

In order to study the effect of cyclones on multi-storied coastal structures, four different structures (3-, 6-, 12- and 24-storied building) are used for structural analysis. Similar beam size (10″ × 20″) is used in all three buildings. But the same size of columns is not used; i.e., column sizes are changed at every 6 stories. Three different sizes of columns (Center, Edge and Corner) are designed for every 6 stories.

The models used are 60-ft (3 bays of 20′ each) in the long direction and 40-ft (2 bays of 20′ each) in the short direction. Uniform story height of 10-ft is used for all buildings. The structural floor plan is shown in Fig. 37.

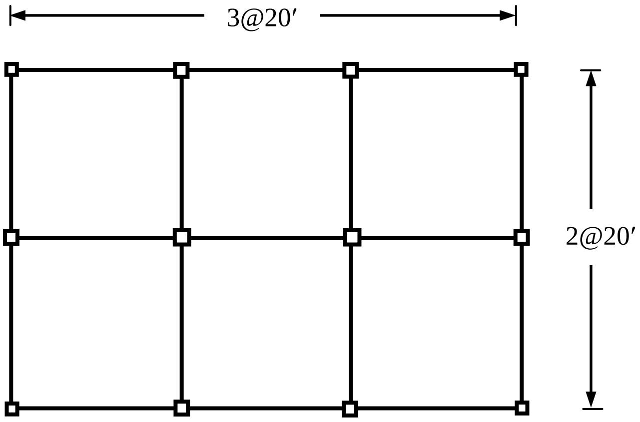

Fig. 37: Building Lay-out Plan

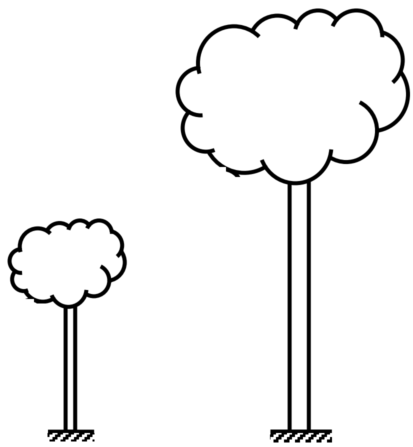

Fig. 38: Elevation of the 20-ft and 50-ft trees

The structural response of protective vegetation is simulated using two tree models (of diameters 10″ and 20″ for heights 20′ and 50′ respectively, shown in Fig. 38), subjecting them to hydrodynamic loads.

4.2.1 *Results from Numerical Analysis*

Moment-Curvature Relationship

Fig. 39 shows the moment vs. curvature relationship of the central column of 12-storied building (shown here as an example), as well as the larger tree (50-ft high with a diameter of 20-in) used as protective vegetation. These relations are used in the subsequent nonlinear dynamic analyses of the structures as well as the protective vegetation subjected to wave, current and wind loads.

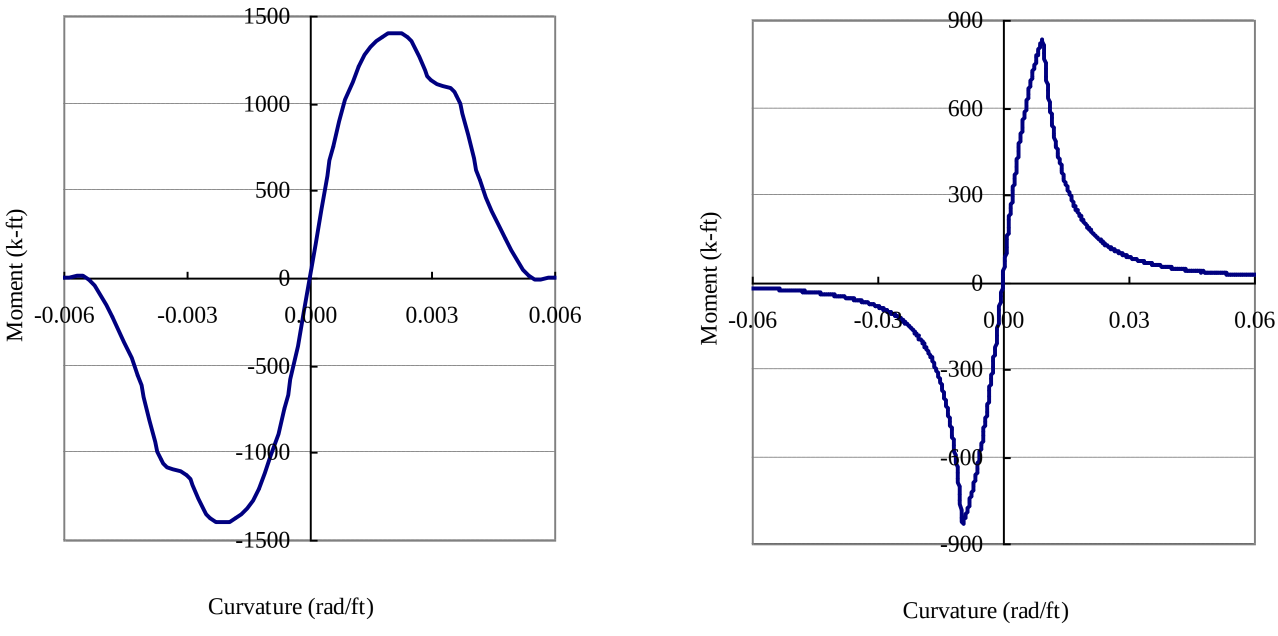

Fig. 39: Moment-Curvature relation for (a) Central Column of 12-storied Building, (b) Larger Tree

Deflections for Wave, Current and Wind Loading

Figs. 40(a)~41(b) show the top floor deflections of the buildings for wave, combined wave-current and wave-current-wind loads [Fig. 32 for 3- and 6-storied while Fig. 33 for 12- and 24-storied building]. Moreover, each figure here has three plots, showing the corresponding relations for wave loads only (W), combined wave-current loads (WC) and wave-current-wind loads (WCW).

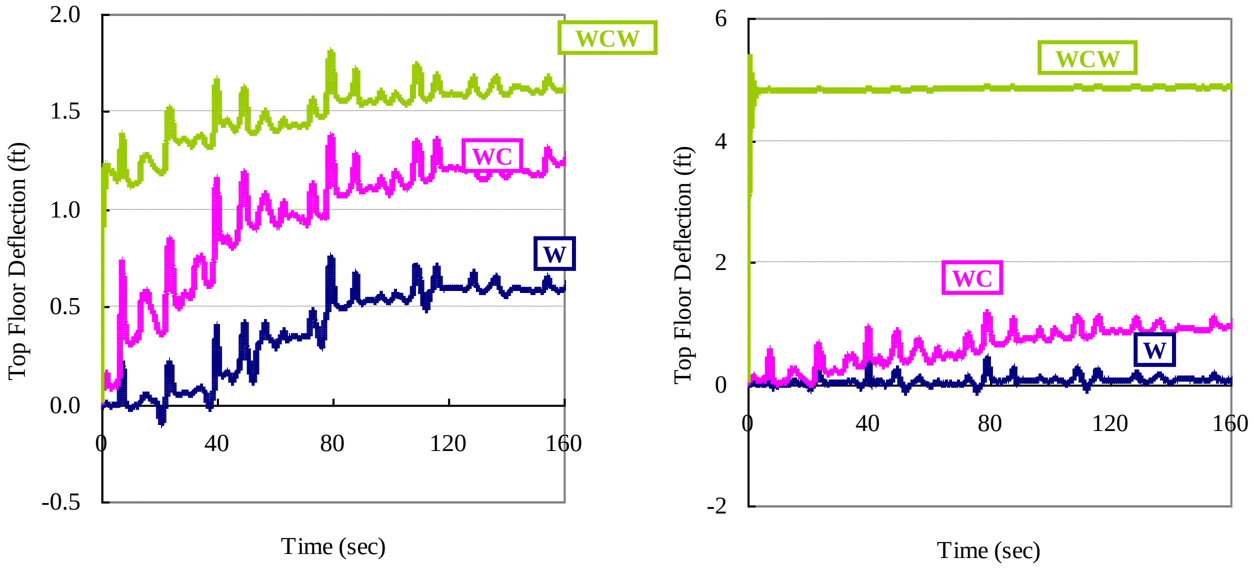

Fig. 40: Top Floor Deflections of (a) 3-Storied Building, (b) 6-Storied Building

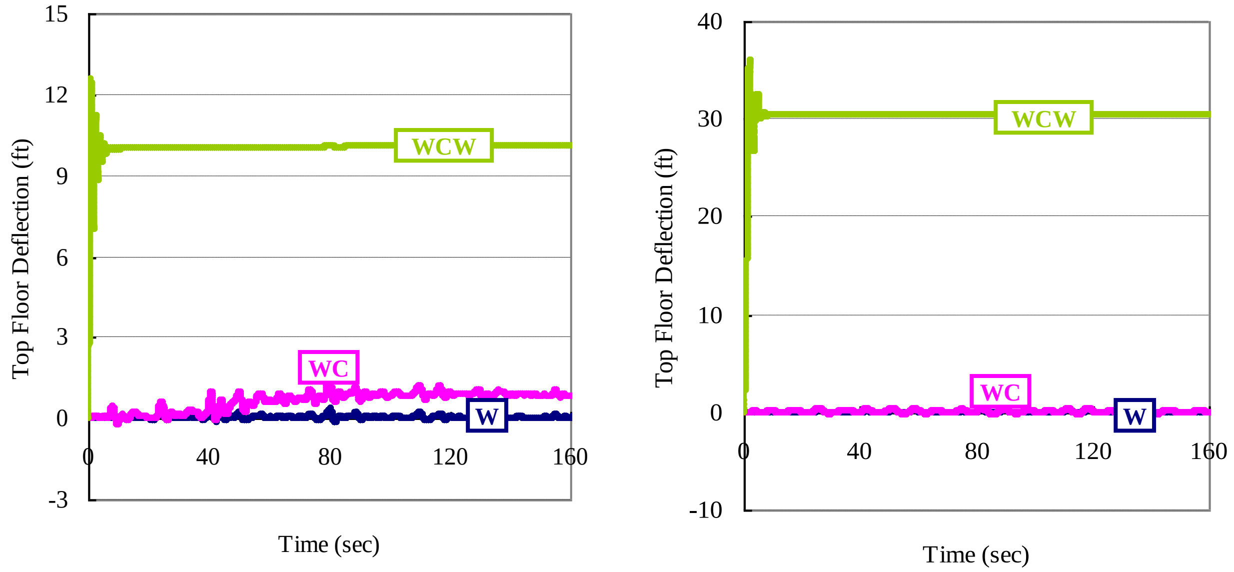

Fig. 41: Top Floor Deflections of (a) 12-Storied Building, (b) 24-Storied Building

The results show the very significant effect of wind loads on the structural deflections, which gets even more pronounced for the taller buildings (but is not too important for 3-storied building). This is because the action of the wind loads being close to the tip of the structures, leading to more significant effects on the structural deflections. Such large deflections are definitely not permissible for the structures, and alternative structural systems (e.g., larger beam and column sections, shear walls, bracings, etc.) are to be used to reduce the deflections and improve serviceability of the buildings.

The variation of ground floor column curvatures [Fig. 42(a)~43(b)] also demonstrates the very significant effect of the wind loads and also shows that the designed structures not only fail the serviceability criteria but also are not safe. The very large curvature requirements often far exceed the ultimate curvature capacities of the sections (shown earlier by the M-ϕ relationships).

Therefore, in addition to the structural measures suggested to reduce deflections, the sectional ductility need to be improved using special detailing measures.

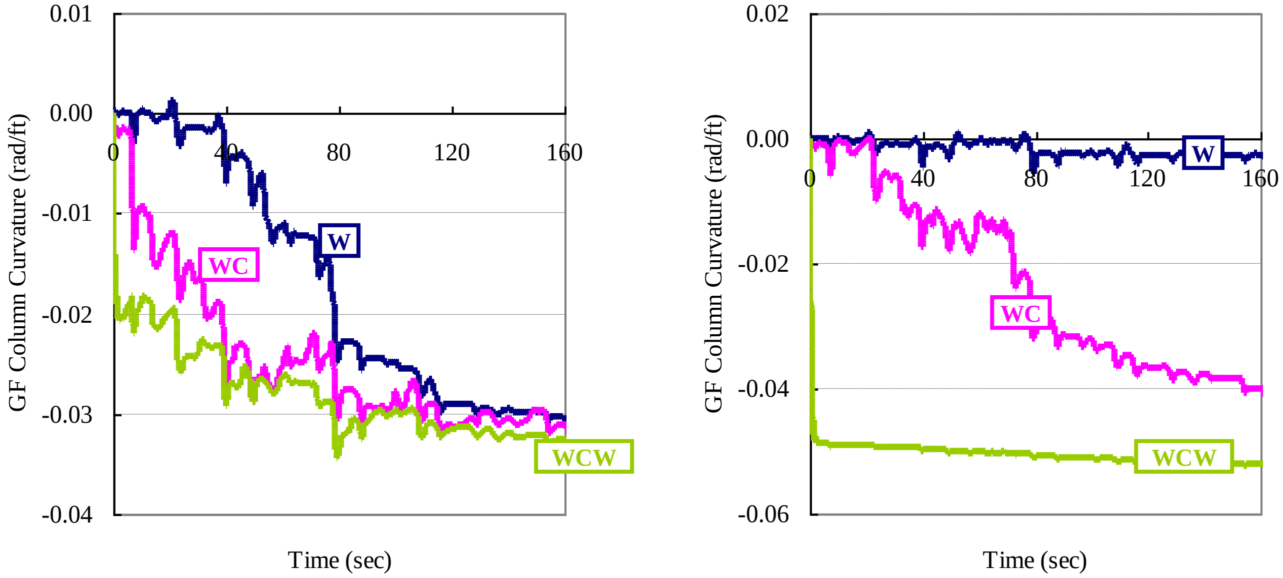

Fig. 42: Ground Floor Column Curvatures of (a) 3-Storied Building, (b) 6-Storied Building

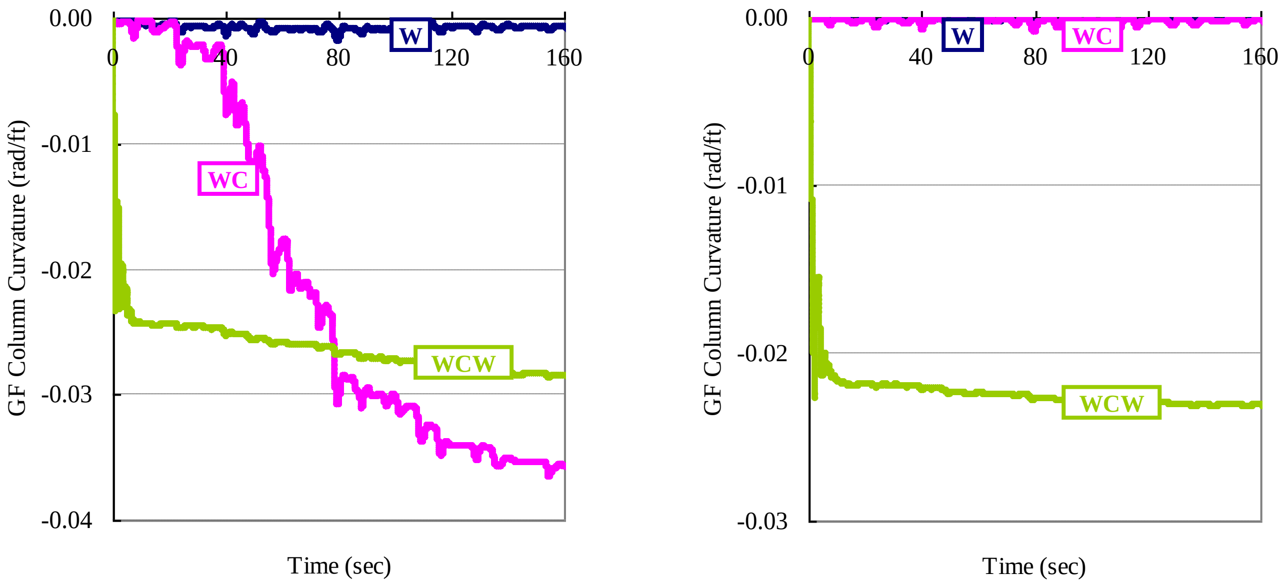

Fig. 43: Ground Floor Column Curvatures of (a) 12-Storied Building, (b) 24-Storied Building

4.3 Dynamic Response of RC Railing to Vehicular Impact (Susmita & Anam 2012)

Conventional design of traffic barrier or bridge railing involves assumption of distributed static loads to account for crowded human loads. In addition, it includes concentrated static loads at selected critical locations to approximate the effect of vehicular impact. However, such design of highway bridge railing is often insufficient to sustain the dynamic impact of vehicles. The railing may not be able to absorb the horizontal loads created by the impact of a truck, which could lead to a serious damage. Conventional strengthening methods, such as increasing the area with concrete, are unsuitable because of lack of space and for aesthetical reasons. In this work, the arrangement shown in Fig. 44 is chosen for the railing and rail post, while Fig. 45 shows the cross-sections of railing and post obtained from RC design.

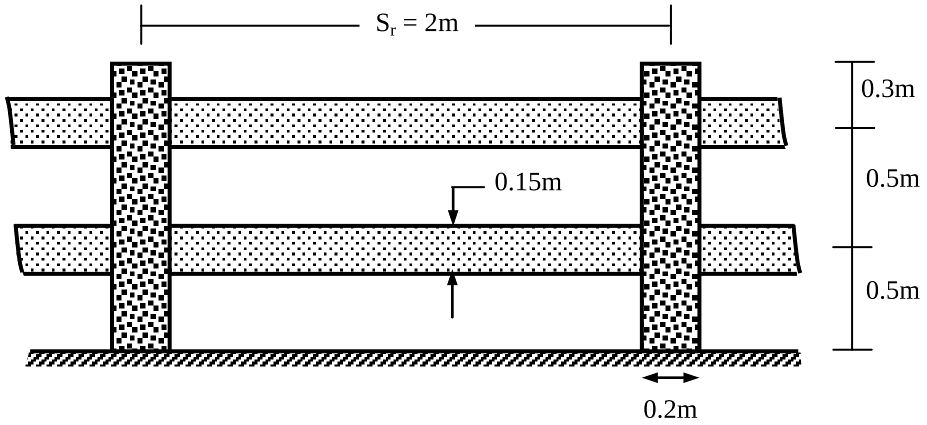

Fig. 44: Elevation of RC Railing used in the analyses (showing c/c view only)

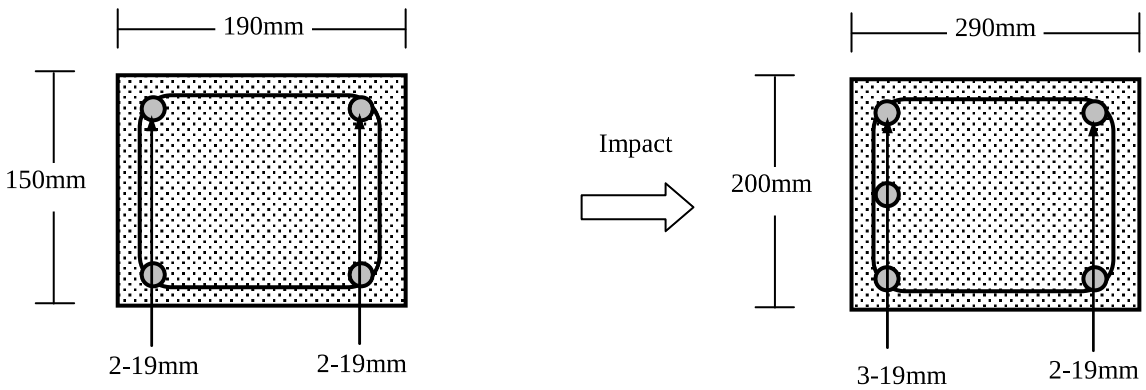

Fig. 45: Cross-sections of Railing and Rail Post

The nonlinear moment-curvature relationship of the railing and rail post sections are derived numerically and shown in Fig. 46 for different strain rates (i.e., static load and strain rate = 100/sec). It demonstrates the strongly nonlinear properties of both the sections, which are modeled as elastic-fully plastic. Also apparent is the influence of strain rate on the cross-sectional properties of both sections. Most significantly, the consideration of strain rates makes the sections stronger. Apparently, the effect on ultimate ductility of the section is not very significant because it depends on the ductility of steel, which is not much affected by strain rate.

Nonlinear structural dynamics is used to analyse the bridge railing shown in Fig. 44. Parametric studies are performed to demonstrate the effects of different items on the dynamic response of the structure (represented by rail post deflections) to vehicular impact. A significant component of the vehicular velocity being perpendicular to the structural plane, this is to simulate the effect of a fast moving derailed vehicle colliding with the bridge railing. A 2-ton car colliding at a velocity of 100 kmph and angle 30° is taken as reference for vehicle, while the reference damping ratio of the structure is assumed 8%. Sample results of the parametric studies are shown in Fig. 47(a) to Fig. 47(b). It involves the variation of vehicular weight from 2 tons to 1 and 4 tons, vehicular velocity from 50 to 100 kmph and angle of impact from 30° to 90°. The results (deflection of rail post) are summarized in Table 10, showing the possible collapse of the bridge railing (exceeding the allowable top deflection of 250-mm) in most of the cases studied.

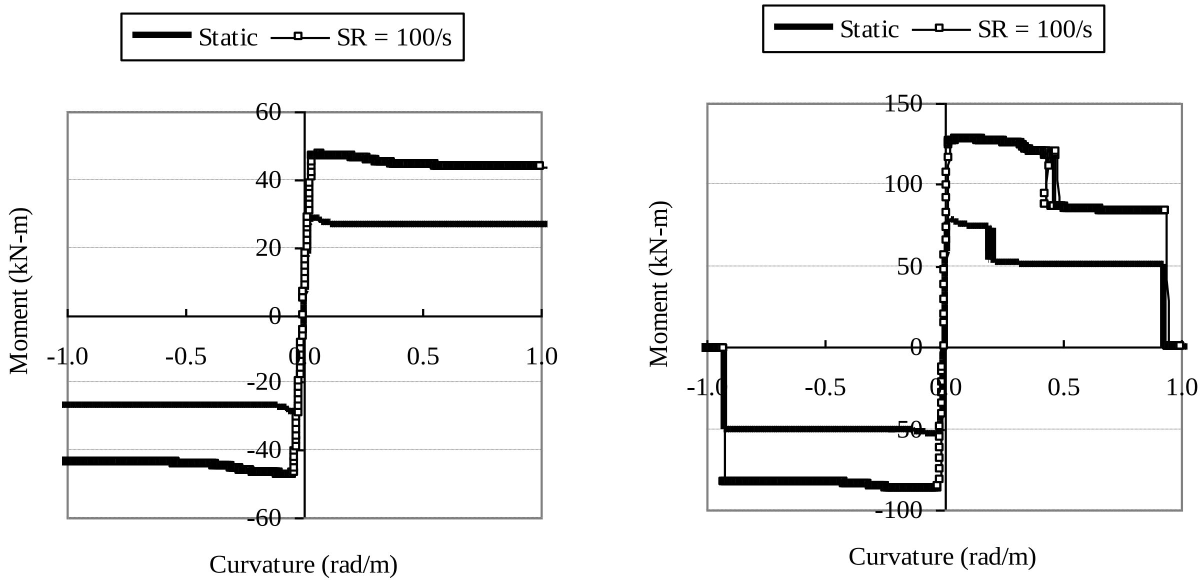

Fig. 46: Moment-curvature relationship of Railing and Rail Post for different strain rates

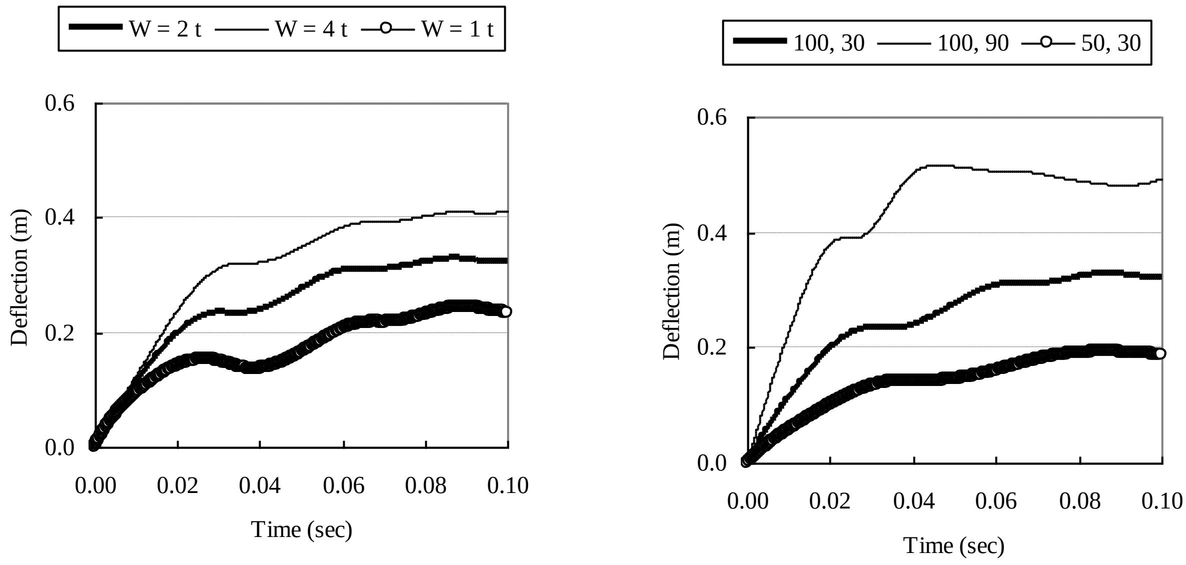

Fig. 47: Dynamic Response showing effect of (a) Vehicular Weight, (b) Velocity and Angle

Table 10: Maximum Deflections (mm) from Parametric Studies

Δ_{ult}	Δ_{Ref} of various Posts			Damping Ratio		Weight (ton)		Velocity (kmph), Angle (θ)	
	Top	Middle	Side	4%	2%	4	1	100, 90°	50, 30°
250	330	168	187	377	390	413	244	517	193

5. CONCLUDING REMARKS:

The paper has presented an overview of the performance of common structures subjected to various types of static and dynamic loadings, and the role of ductility in improving it. The presentation of the performance of common structures and possible measures to improve them basically follows a literature review approach, in that assessments and illustrations from different literatures are cited. The final portion of the paper presents conclusions from some research works at UAP on this topic. The following may be mentioned as some of the conclusions from this work.

* Overloading of structural members like slab, beam and column and poor construction practice (resulting in under-strength materials) have caused several catastrophic structural failures worldwide, including Bangladesh. A more careful assessment of the structural loads, adherence to rules and better construction practice is necessary to avoid such tragedies. Retrofit measures like beam/column jacketing and various types of confinement may be used to improve structural performances.
* Differential support settlement may turn out to be the cause of very serious structural damage in Bangladesh, particularly in view of the poor sub-structure conditions induced in recent structures. Proper assessment of soil properties from accurate soil testing, coupled with measures of soil strengthening may avert structural distress and collapse due to support settlement.
* Impact loads may also cause catastrophic structural failures like progressive failure of slabs or failure of bridge railing due to vehicular impact. Various member detailing measures and shock-absorbing devices can be used to improve the structural performance.
* Long-term machine vibrations may cause structural fatigue/distress and eventually lead to collapse. They should either be transferred to very rigid sub-structures or supported on flexible springs/dampers to reduce the potential threat.
* Blast load may become a threat to structures and overall stability of the society, and should therefore be taken care of with adequate measures (e.g., large stand-off distance, shock absorbers and member ductility). The possible use of blast load in controlled demolition of buildings should also be investigated.
* Measures to resist cyclonic storms (a combination of wave, current and wind forces) also include adequate protective vegetation and member ductility measures.

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