Addis Ababa Institute of Technology (AAiT.)

School of Civil & Environmental Engineering.

Stream of Structural Engineering

BEHAVIOUR OF CONVENTIONALLY DESIGNED AXIALLY LOADED REINFORCED CONCRETE RECTANGULAR COLUMNS SUBJECTED TO LATERAL IMPACT LOAD.

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Table of Contents

1-Acknowledgement

First I would like to thank the Ethiopian Roads Authority (ERA) for providing this chance and Dr. Girma Zerayohannes for his support and consultation. I also like to thank Addis Ababa institute of Technology (AAiT) and my family.

2-Abstract

Reinforced concrete buildings elements mostly designed for loads from live, dead and lateral load from wind and earthquake. But due to the use of basement floors for car parking and garage purpose the internal columns are susceptible for unforeseen accidental load arise from vehicular impact. This research presents the study on the response of reinforced concrete columns which are designed for axial load case due to gravity only when subjected to accidental vehicular impact load using Sap2000v14.Even the loading condition is dynamic the equivalent static impact load is taken 75KN in our case according to Euro code for accidental load case and the impact height is 0.75m from bottom of the column. The design parameters considered includes the vertical steel ratio, concrete cross sectional area, axial load, different concrete and steel grade. The PMM ratio (capacity ratio) and shear capacity result indicates that columns with higher concrete sections are safer and also columns with lesser longitudinal steel ratio safer than with equivalent (axial load carrying) column section with higher longitudinal steel ratio which are designed to carry the same axial load. Additionally sections designed to support higher axial load (supporting more and more stories) are safer when compared to those supporting lesser axial load.

3-Introduction.

3.1-Background of the study.

The increased demand for land and its higher cost has created the need to provide building's basement for parking and garage purpose. This intern brought susceptibility of concrete columns to vehicular impact. The behavior of columns subjected to lateral impact is an interest and different experimental and finite element analysis has been conducted.

Columns are primarily designed to support the gravity load from the structure above them and the moment created due to eccentricity and unbalanced moment form the adjacent beam.

There are few experimental investigations and finite element analysis conducted on laterally impacted columns that shows the effect of strain rate and confinement particularly under mid span hard impact condition.

In actual condition the initial kinetic energy of the colliding object can be transferred into many other forms of energy and into elastic-plastic deformation or fracture of the structural elements in both the building structure and the colliding object (fig,3-1).

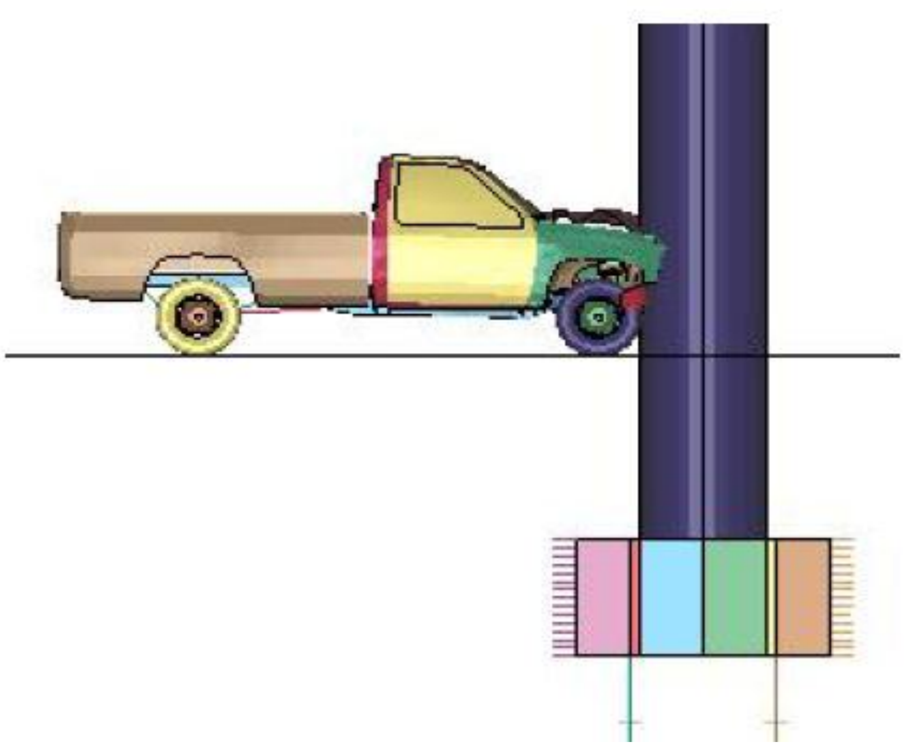

Figure 3-1: car collide with a column structure [14].

The real impact condition is soft or in between soft and hard however hard impact represents the upper bound of typical vehicle impact and fraction of the initial kinetic energy only transfer to the structure.

Experimental findings shows increase in the rate of loading usually increases both the compressive strength of concrete and the yield strength of steel generally the strength of the material as whole when it loaded dynamically. Hence it may be expected that the moment capacity of reinforced concrete columns increases with increasing in the loading rate.

The different literatures referred demonstrated only for the case when a single column faces impact load from falling mass but in this thesis the three dimensional conventionally designed frame with different stories has been analyzed for equivalent static impact load stated in euro code for parking and garages.

The sample columns have been selected for different parameters like concrete cross sectional area, longitudinal steel ratio and concrete and steel strength class according to the British code.

3.2-Problem statement

The main concern in designing reinforced concrete columns is to accommodate the load from the structure above them rather than the impact of lateral load from moving vehicles. But currently there is a low that enforce buildings near the roadside to provide one car parking area for each $70m^2$ floor area.

Currently the cost for land is higher and it's not economical to provide it in open land. Then it makes the need to use basement floors for parking and due to this it's obvious that internal columns to face damaged due to moving vehicles even if they are not designed for this accidental load case.

So it's necessary to see the effect of the impact and to recommend the research findings.

3.3-Objective.

The objectives of this study are:-

To show the effect of accidental impact load on the conventionally designed reinforced concrete columns.

3.4-Hypothesis.

Does conventionally designed rectangular RC columns in basement floor of the buildings are enough to support the accidental load due to vehicular impact.

4-Litreture Review

4.1-Characterizations of loads.

Loads can be characterized based on their duration compared with natural frequency of the impacted member. The three general cases for relative relationships between the load function and the structural response as shown in Figure 4-1. If the load is over before the structure reaches its maximum response it's in the impulsive domain. If the structure reaches its maximum deflection well before the load is over it's in the quasi-static domain. Meanwhile, if the maximum deflection is reached near the end of the loading it's in the dynamic domain. Typically, impact is considered as impulsive loads because the loading occurs very rapidly and prior to the structure fully responding [16].

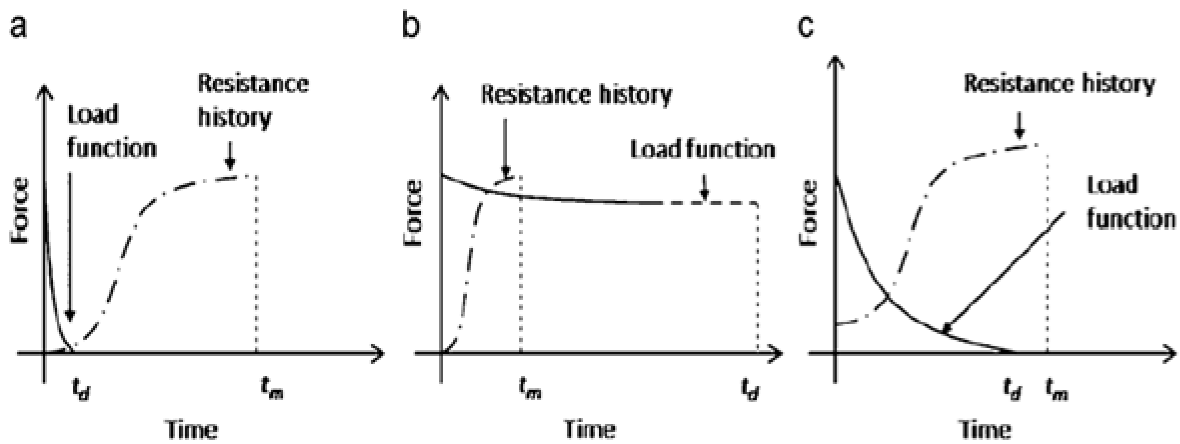

Figure 4-1: Typical response domain: (a) Impulse; (b) Quasi-static; and (c) Dynamic [16]

4.2-Characterization of Impact loading.

The behavior of structures under impact is complicated by many parameters including the effect of higher modes of vibration, changes in the failure mode due to propagating stress waves, and localized damage and its effect on overall strength and stability. Impact is generally classified as soft impact and hard impact.

Soft impact occurs when the kinetic energy of impact is mostly absorbed by plastic deformation in the striking body. Impact velocity is generally low and there is an eligible propagation of stress waves in the impacted body. Failure mechanisms in the impacted body are similar to those associated with static loading.

On the other hand, hard impact occurs when the kinetic energy is almost completely absorbed by the struck body. Here, the striking body barely suffers any deformation. Impact velocities are generally high and complicated stress waves propagate through the struck mass leading to failure.

Based on these definitions, vehicular impact with reinforced concrete structures can be generally categorized as soft/hard since damage can simultaneously occur in both vehicle and pier [12].

4.3-Impact load on structure.

The mechanics of a collision may be rather complex. The initial kinetic energy of the colliding object can be transferred into many other forms of kinetic energy and into elastic-plastic deformation or fracture of the structural elements in both the building structure and the colliding object. Small differences in the impact location and impact angle may cause substantial changes in the effects of the impact. This, however, will be neglected and the analysis will be confined to the elementary case, where the colliding object hits a structural element under a right angle.

Even then, impact is still an interaction phenomenon between the object and the structure. To find the forces at the interface one should consider object and structure as one integrated system. Approximations, of course, are possible, for instance by assuming that the structure is rigid and immovable and the colliding object can be modeled as a quasi-elastic single degree of freedom system (see Figure 4.2). In that case the maximum resulting interaction force equals:

$$F = v_r\sqrt{(k.m)} \qquad Eq-4.1$$

v_r = the object velocity at impact
k = equivalent stiffness of the object
m = mass of colliding object
In most cases the colliding object will respond by a mix of elastic deformations, yielding and buckling [1].

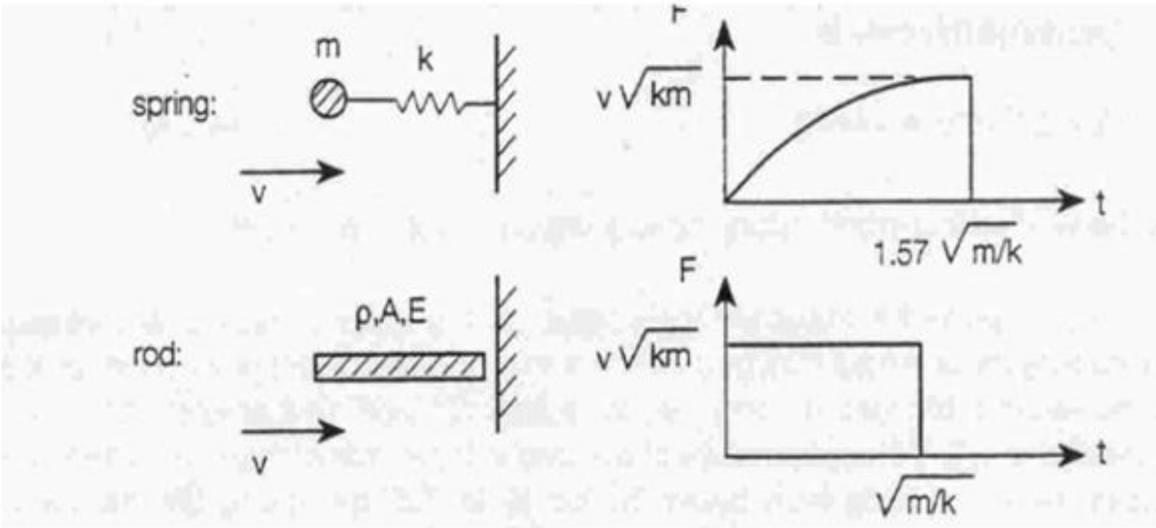

Figure 4-2: Spring and rod models for the colliding objects source:[1]

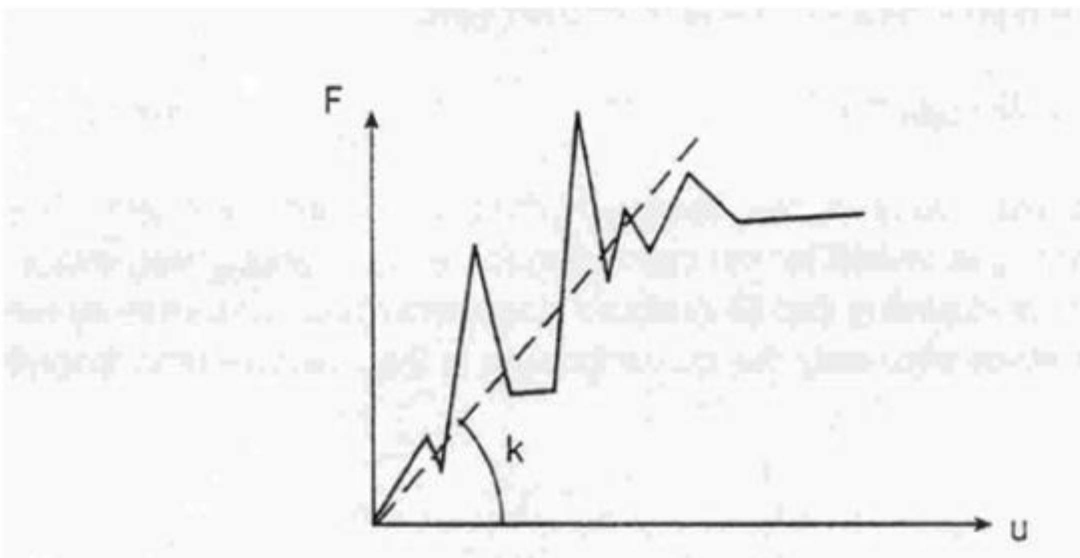

Figure 4-3: General load displacement diagram of colliding object Source: [1]

4.4-Effect of Impact duration.

Impact duration is an important parameter which depends on the properties of the column and impacting vehicle. The impact duration can vary from 50ms to 150ms in most cases and the duration was varied in this range to investigate its influence. Numerical simulations revealed that columns that withstood 150ms impact may collapse under 50ms impact even though the mass and velocities of the vehicles are the same (ie same impulse). At the next stage, the peak forces of the impact pulses were adjusted until the iso-damage condition was achieved. The peak forces (at near collapse) were quite similar irrespective of the durations and hence the peak force defined the vulnerability of the column to vehicular impact. Consequently it was identified that the vehicle impacts can be located in-between dynamic and quasi-static regions of the pressure impulse diagram where neither the shape nor the duration of the impact has considerable influence on the vulnerability of the columns. The average impact duration of many medium-sized vehicles is 100ms. [7]

4.5- Strain Rate Sensitivity.

Typical impact tests result in strain rates ranging from 10^{-1} to 10^{2} s^{-1} (Figure 4-4). [Note: Quasi-static loading is typically about 10-5 s-1]. It is important to understand how this elevated loading rate will affect the response of the structure as a whole and the materials that constitute the structure. It has been well documented that materials and structures display apparent strength gain during dynamic and impulsive testing when compared to quasi-static experiments. This apparent change in material behavior related to changes in rate of loading is called strain rate sensitivity.[9]

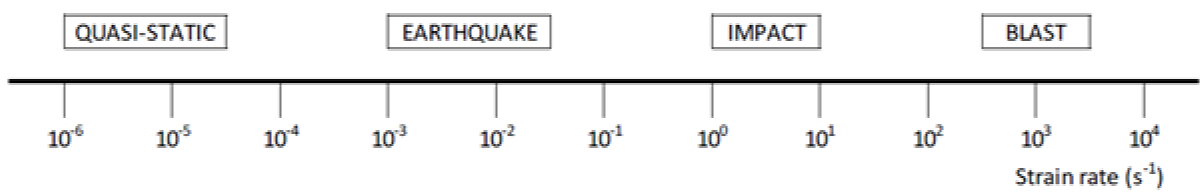

Figure 4-4: Strain rates associated with different types of loading [9]

The fundamental cause of this gain in strength in dynamic and impulsive testing regimes is still uncertain and there exists no consensus within the scientific community to explain it [9].

4.6-Strain rate.

Over the past decades numerous works have been conducted on both theoretical and experimental studies of the strain rate effect on cement-based materials. Most of the dynamic failure criterions are obtained by the curve fitting of experimental data, and conversely, the physical mechanism and the mathematical expression of the strain rate effects is still far from fully understood. Nowadays, two approaches to describe dynamic behavior of cement based materials are commonly accepted, and they are:

1) Critical dynamic strength criterion, which handles dynamic failure strength of a cement based material by extension of material's static strength by a dynamic increase factor (DIF); and
2) The inertial effect and wave propagation mechanisms, which believes the dynamic failure strength of a material is not an intrinsic material property, but a computational characteristic.

Cotsovos and Pavloviæ study on strain rate effect by using a quasi-static constitutive model as well as static material strength, they performed non-linear finite element analysis to study the behavior and failure of concrete specimens under dynamic load, and found that their results from numerical simulations were consistent with the published experimental data. Based on their results they concluded that the effect of strain rate on the specimen's behavior reflects the effect of the inertia loads instead of an intrinsic material property. Therefore, the use of experimental data from dynamic tests to model constitutive behavior of concrete under dynamic loads is questionable [5].

However, on the other hand, as a convenient format for implementation into numerical models DIF is still widely accepted to represent the strain rate effect on the dynamic behavior of concrete. Particularly in this study, the DIF is still employed to represent the strain rate effect. This is because that both the SHPB test to investigate the strain rate effect, which is employed in this study to find the DIFs, and the projectile impact tests to study the concrete target responds to high velocity projectile impact, which are employed as real cases to be simulated in this study, were performed by the authors by using fiber reinforced concrete materials having similar properties [5].

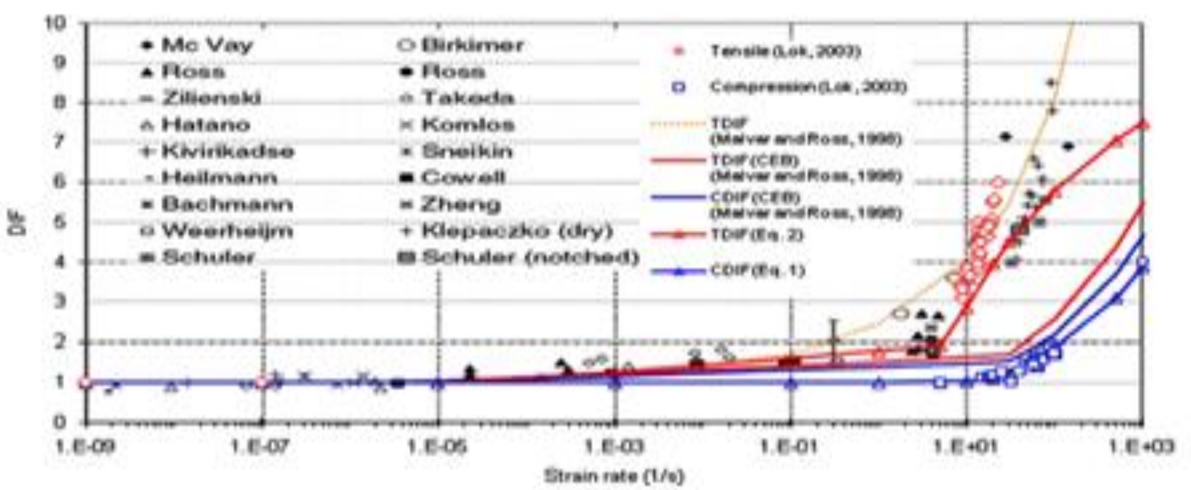

Figure 4-5: DIF for tensile and compressive strength[5]

To explain the strength increase of materials when loaded dynamically, Cotsovos and Pavlovic (2008) presented a two-part study that constituted a major departure from recent thinking regarding material modeling of concrete under high rates of strain. A simplified FE model was employed to investigate the effect of loading rate on the behavior of prismatic concrete elements under high rates of uniaxial compressive loading in an effort to identify the fundamental causes of experimentally observed apparent strength increase of specimens when a threshold range of loading rates is surpassed. The authors found that it is inertial effects that cause the strength gain [16].

Any increase in the rate of loading usually increases both the compressive strength of concrete and the yield strength of steel. Hence it may be expected that the moment capacity of reinforced concrete columns increases with increasing in the loading rate. Figures (4-6 and 4-7) present the moment-curvature relationships derived analytically under different rates of loading. These rates are 0.0001/sec (a typical quasi-static value), 0.001/sec, 0.01/sec (typically expected under seismic loads) and 0.1/sec (a typical value under impulsive loads):

1. For different yield strength of the main reinforcement the curvature ductility factor under the strain rate of (0.1/sec) increased an average by about (80%) over the curvature ductility under static load.

2. For different yield strengths of the transverse reinforcement the curvature ductility factor under the strain rate of (0.1/sec) increased an average by about (50%) as compared to the static loading, Fig.(4-10-b,e).

3. For different concrete compressive strengths the average increase in curvature ductility under the strain rate of (0.1/sec) is about (35%) compared to the static loading, Fig. (4-10-c).

4. The average increase in curvature ductility under the axial force in the range of (0.1 fc'Ag) to (0.9 fc'Ag) is about (25%) for the strain rate of (0.1/sec) compared to the static load condition for which the strain rate is (0.0001/sec), Fig. (4-10-d).

Figs.(4.6-4.9) show the effect of strain rate on the moment curvature relationships of the column section for different analysis parameters. Under an axial load of (0.6 fc'Ag) and for different fy, fc', fyt and ties spacing the average increase in moment capacity at strain rate of 0.1/sec over the static rate is (35%), Figs. (6,7,8 and 9). This increment has been found to vary in the range of (20%) to (80%) for the axial loads (0.1 fc'Ag) and (0.9 fc'Ag) respectively, Fig. (9) [2].

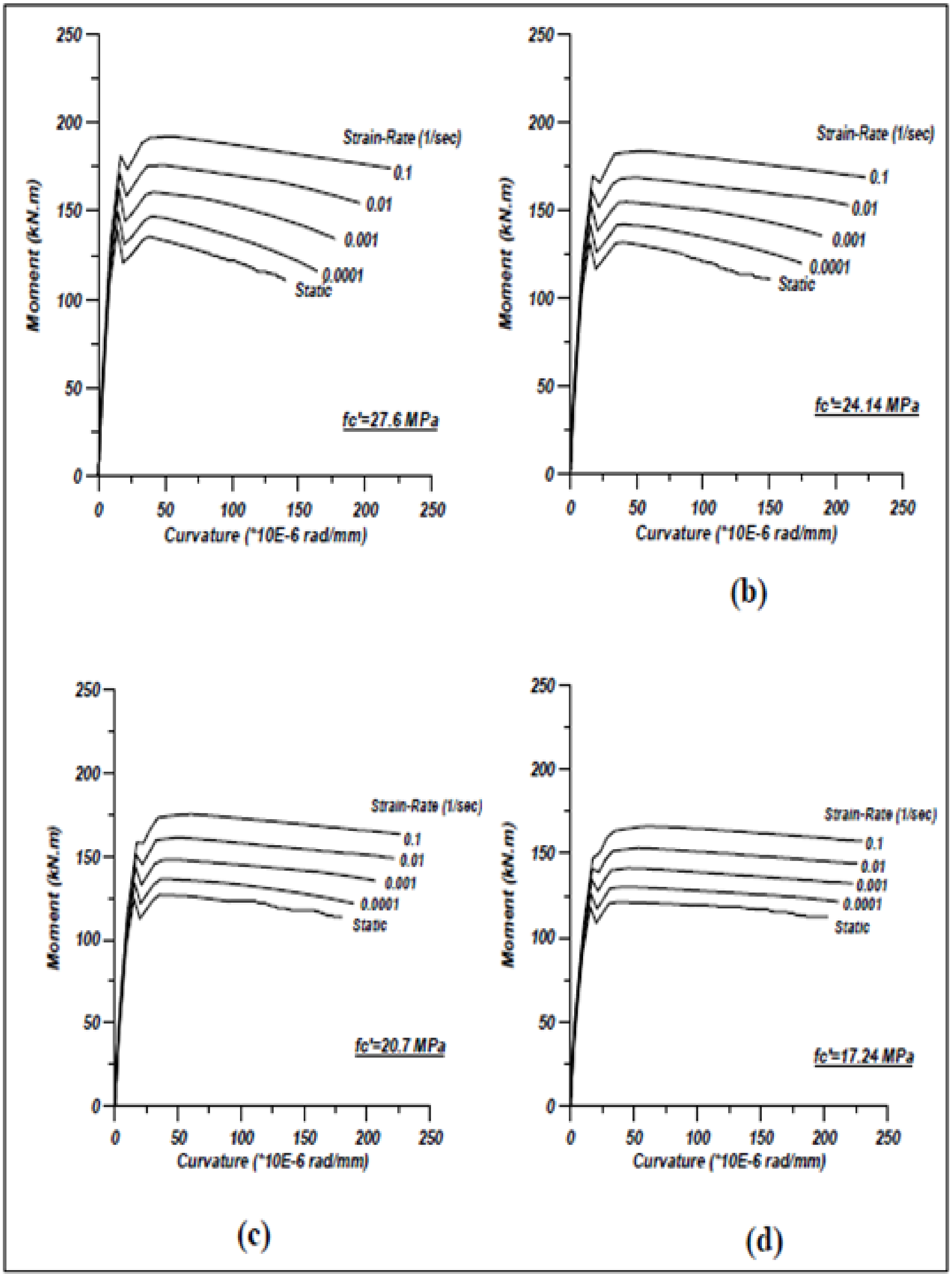

Figure 4-6: Effect of strain rate on moment curvature relationship for different compressive strength of concrete f_y=400Mpa, f_{yt}=276Mpa, P=0.6f'_cAg

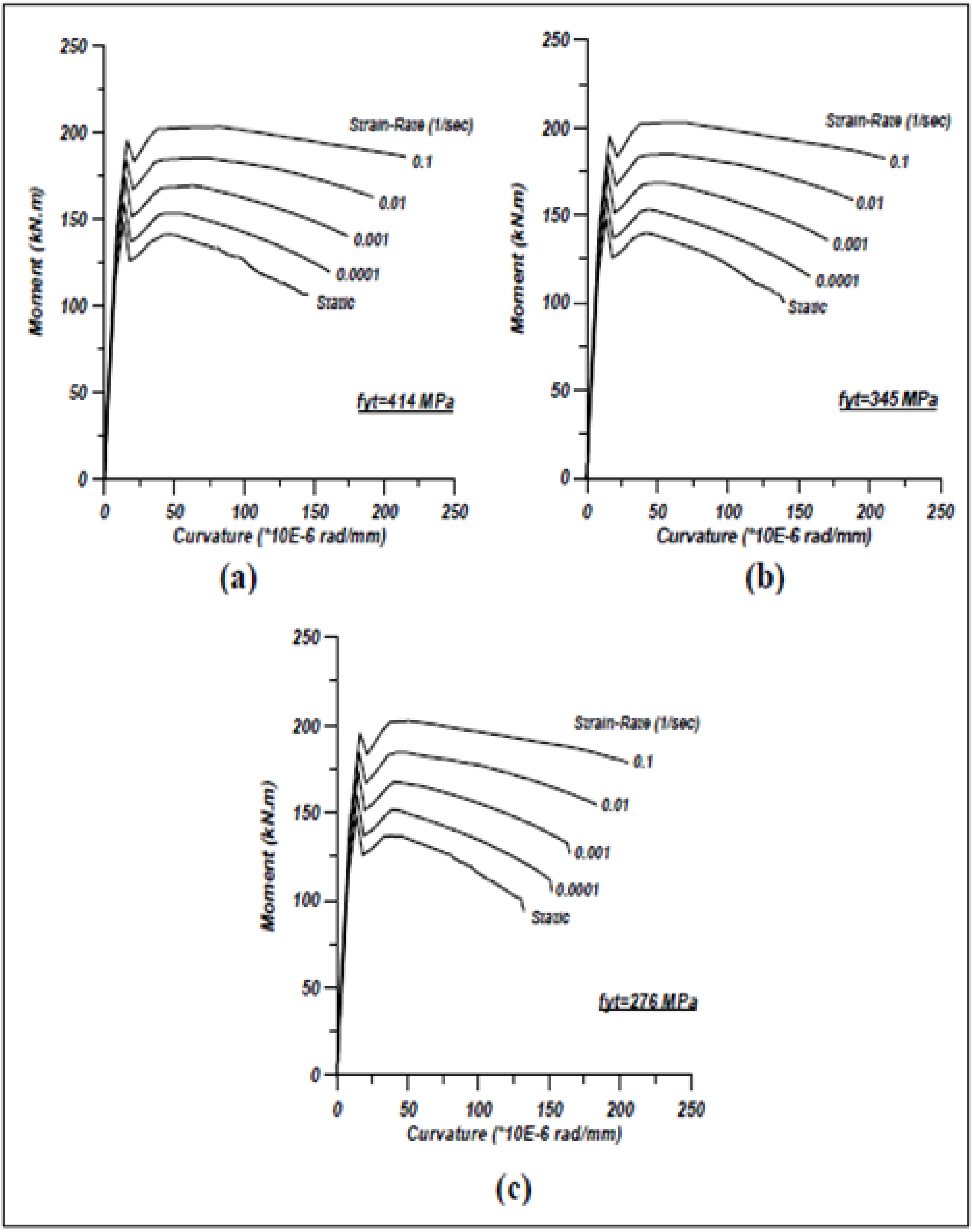

Figure 4-7: Effect of strain rate on moment curvature relationship for different yield strength of transverse reinforcement f_y*=400Mpa, f'c=31.8Mpa, P=0.6*f'_c*Ag*

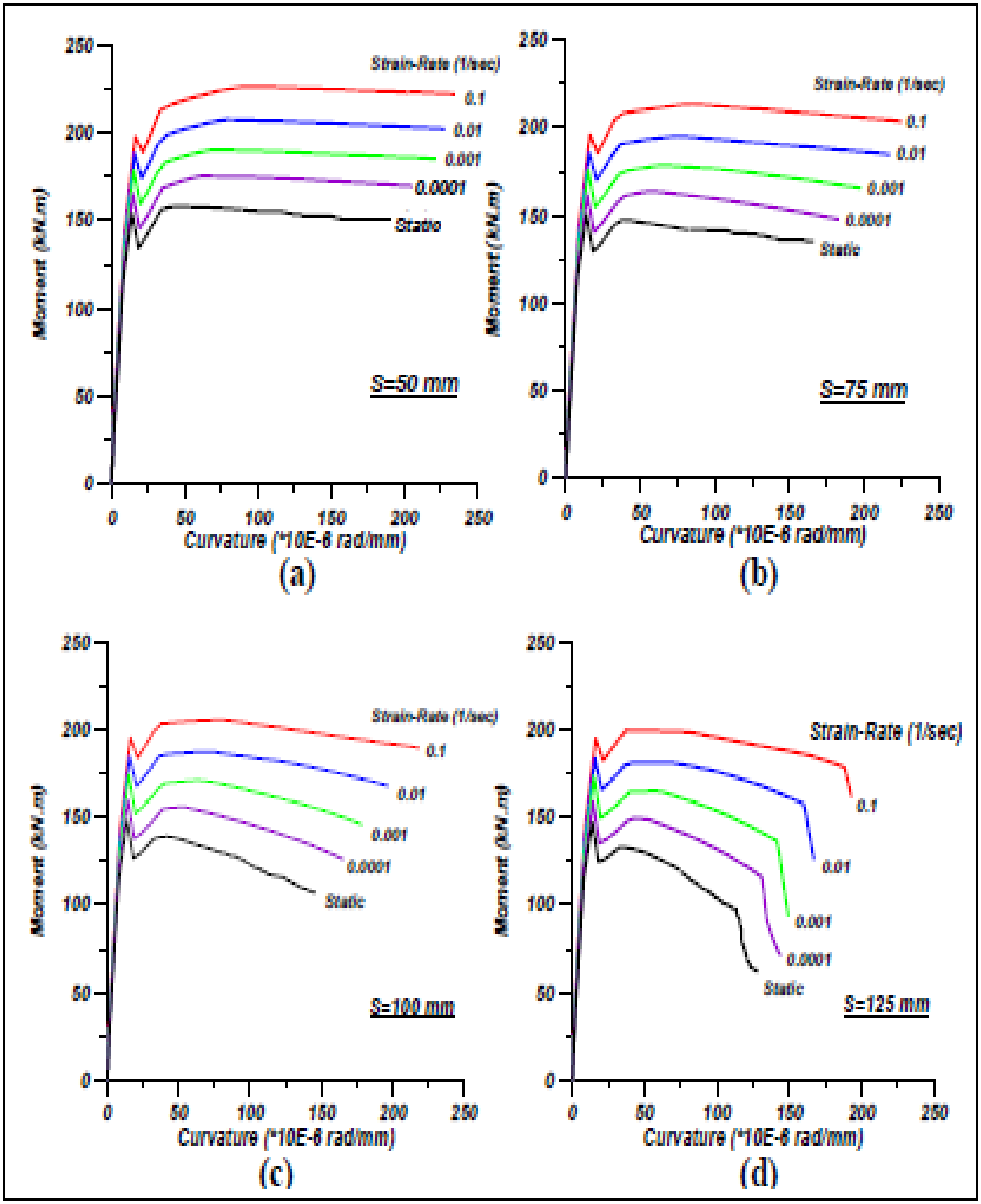

Figure 4-8: Effect of strain rate on moment curvature relationship for different spacing of transverse reinforcement f_y=400Mpa, f'c=31.8Mpa

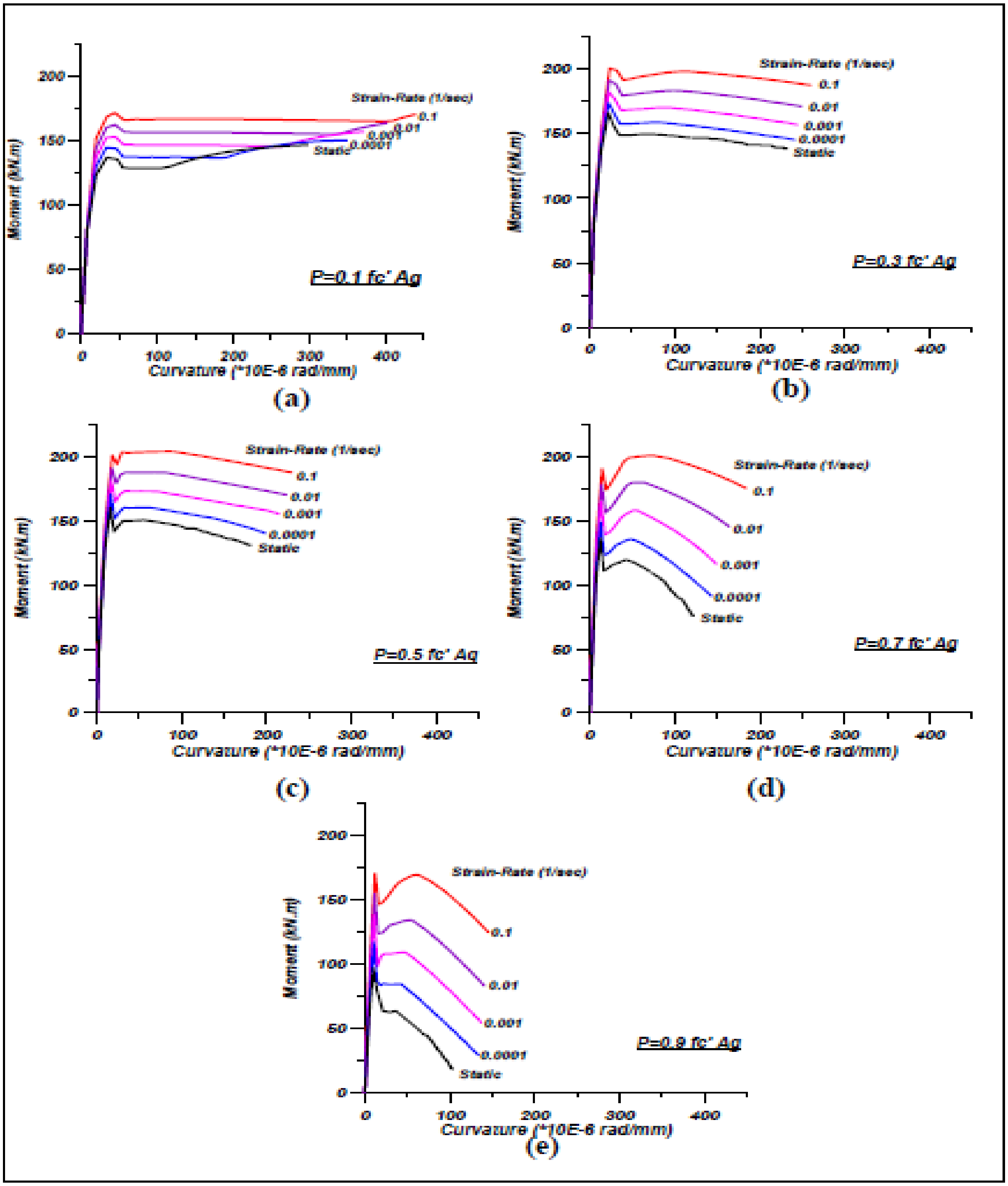

Figure 4-9: Effect of strain rate on moment curvature relationship for different axial load f_y=400Mpa, f'c=31.8Mpa, Fyt=276Mpa

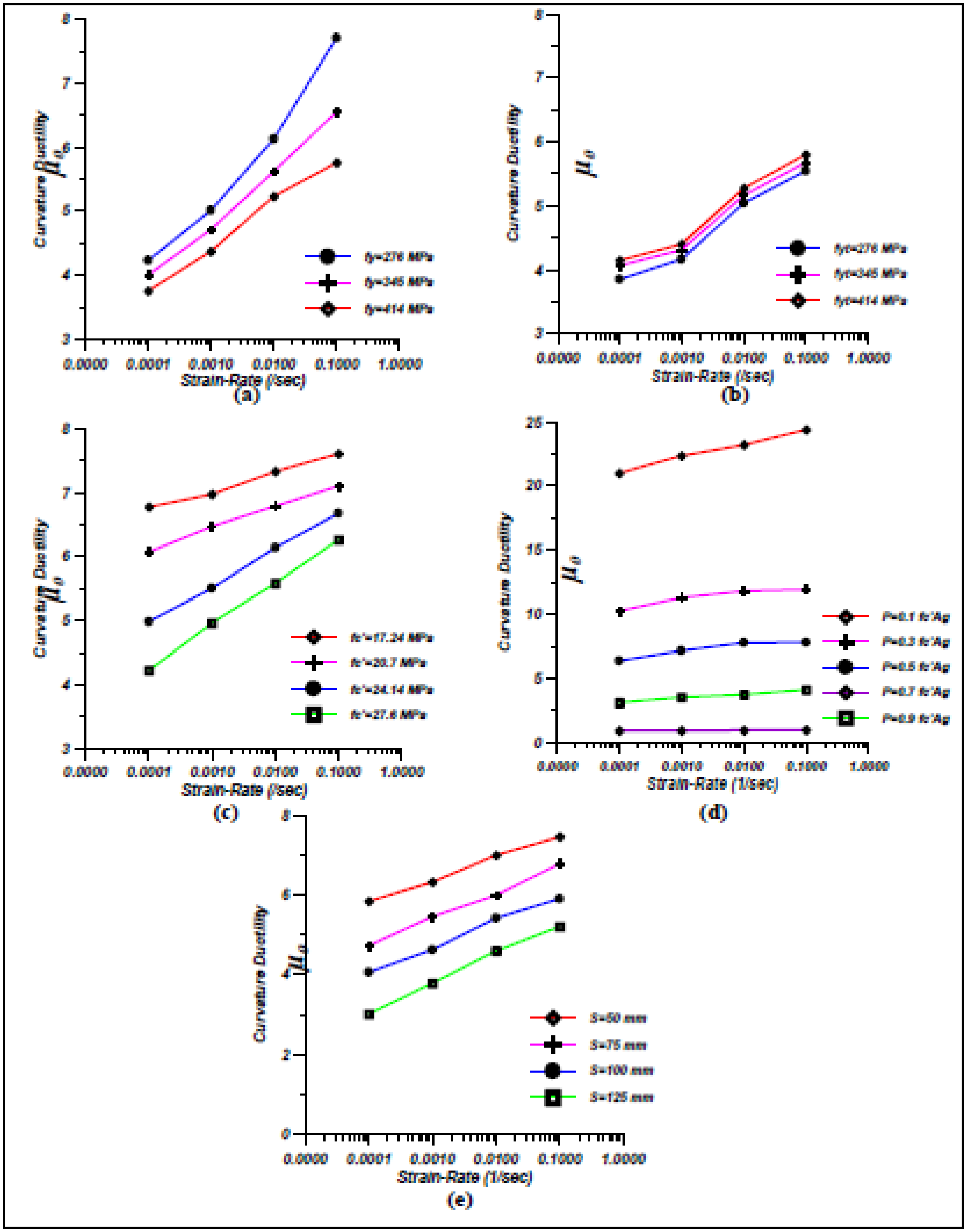

Figure 4-10: Effect of strain rate on curvature ductility for different:

(a) Effect of steel yield strength for main reinforcement (fc'=31.8 MPa, fyt =276MPa, P=0.6 fc'Ag,S=108mm)

(b) Effect of steel yield strength for transverse reinforcement (fc'=31.8 MPa, fy=400 MPa, P=0.6fc'Ag, S=108mm)

(c) Effect of concrete compressive strength (fy =400 MPa, fyt =276 MPa, P=0.6 fc'Ag, S=108mm)
(d- Effect of axial load (fc'=31.8 MPa, fy =400 MPa, fyt =276 MPa, S=108mm)
(e) Effect of spacing of ties (fc'=31.8 MPa , fy =400 MPa , fyt =276 MPa , P=0.6fc'Ag)

The compressive strength of concrete increases with an increase in the strain rate, while the strain rate effect on the elastic modulus is far less pronounced than that on the compressive strength [14].

For concrete in compression at maximum strain rate of $20s^{-1}$ and increase of the dynamic compression strength of about 50% could be expected [13].

Modeling the behavior of concrete and mortar in conditions of high-speed loading conducted on different experimental facilities: It has been observed for the high-speed curve of concrete (for the cases of compression and tension) that the dynamic strength of the material begins to grow in the range 10^0-10^3s^{-1} of strain rates. On the basis of test data (dynamic strength, strain rate), it can be constructed a high-speed curve that characterizes the dependence of the dynamic strength of strain rate material and combines the material behavior at various levels of quasi-static and dynamic conditions. It should be noted that in fact the appearance of the curve depends on the history of the load; however, as will be shown below, it can be obtained based on the incubation time criterion and its material parameters, which are invariant to any impact history. [15]

4.7-Dynamic increase factor.

The dynamic increase factor (DIF) is the ratio of the dynamic to static strength, is normally reported as function of strain rate. For concrete, the DIF can be more than 2 in compression, and more than 6 in tension.
At high strain rates, the apparent strength of these materials can increase significantly, by more than 50% for the reinforcing steel, by more than 2 for concrete in compression, and by more than 6 for concrete in tension.

4.7.1-Concrete.

4.7.1.1- Compressive strength.

CEB FORMULATION
Dynamic increase factor (DIF) for the compressive strength, is given by:

$$f_c/f_{sc} = \left(\frac{\dot{\varepsilon}}{\dot{\varepsilon}_s}\right)^{1.026\alpha_s} \qquad for\ \dot{\varepsilon} \le 30s^{-1} \qquad Eq-4.2$$

$$= \gamma_s \left(\frac{\dot{\varepsilon}}{\dot{\varepsilon}_s}\right)^{1/3} \qquad for\ \dot{\varepsilon} > 30s^{-1}$$

Where f_c= dynamic compressive strength at $\dot{\varepsilon}$

f_{cs}= static compressive strength at $\dot{\varepsilon}_s$

f_c/f_{cs} = compressive strength dynamic increase factor (DIF)

$\dot{\varepsilon}$= strain rate in the range of 30 x 10^{-6} to 300^{-1}

$\dot{\varepsilon}_s$= 30 x 10^{-6} s-1 (static strain rate)

$log\gamma$= 6.156α - 2

$\alpha_s = 1/(5 + 9\, f_{cs}/f_{co})$

$f_{co} = 10Mpa$

This formulation captures specific material behaviors that are reflected in the following properties:

- the DIF is higher for concretes with lower strengths
- all DIFs are related to a strength measured at a specific "quasi-static" strain rate
- the strength enhancement is different for tension and compression.

4.7.1.2-Tensile strength.

The dynamic increase factor (DIF) for the tensile strength, is given by:

$$f_t/f_{ts} = \left(\frac{\dot{\varepsilon}}{\dot{\varepsilon}_s}\right)^{1.016\delta} \qquad for\ \dot{\varepsilon} \le 30s^{-1} \qquad Eq - 4.3$$

$$= \beta\left(\frac{\dot{\varepsilon}}{\dot{\varepsilon}_s}\right)^{1/3} \qquad for\ \dot{\varepsilon} > 30s^{-1}$$

Where f_t= dynamic tensile strength at $\dot{\varepsilon}$

f_{ts}= static tensile strength at $\dot{\varepsilon}_s$

f_t/f_{ts} = tensile strength dynamic increase factor (DIF)

$\dot{\varepsilon}$= strain rate in the range of 30 x 10^{-6} to 300^{-1}

$\dot{\varepsilon}_s$= 30 x 10^{-6} s-1 (static strain rate)

$log\beta$= 7.11δ – 2.33

$\delta = 1/(10 + 6\, f_{cs}/f_{co})$

$f_{co} = 10Mpa$

4.7.1.3-Effect of strain rate on bond.

Hansen and Liepins concluded that "for all practical lengths of embedment of bonded bars, the increase in load capacity of a bonded bar under dynamic loading over static loading is due only to the increase of steel strength under dynamic loading." Reinhardt concluded that, if the bond strength is linearly related to the concrete tensile strength, under dynamic loading the bond strength would be linearly related to the dynamic concrete tensile strength. Takeda indicated that, under dynamic loading, the concrete strain field around the bar is much narrower and could result in brittle steel fracture. This could be numerically verified, since the dynamic increase factors for concrete in tension are significantly larger than those for reinforcing bars. In summary, it appear that the original bond-slip equations, updated with the enhanced concrete and reinforcing steel strengths at the current strain rate, would still be applicable [3].

4.7.2-Reinforcement bar.

Static and dynamic properties were gathered for bars satisfying ASTM A615, A15, A432, A431, and A706, with yield stresses ranging from 42 to 103 ksi (290 to 710 MPa). The data indicates that the DIF decreases for higher rebar yield stress, and that the DIF is higher for yield stress than for ultimate stress. A simple relationship is proposed that gives the DIF (for both yield and ultimate stress) as a function of strain rate and yield stress.

Under dynamic loading, the strength properties of rebars are known to increase by up to 60% for strain rates of up to 10 s-1, and up to 100% for strain rates around 225 s-1. At these high strain rates, the yield stress can increase by 100%, or more, depending on the grade of steel used. The dynamic increase factor (DIF), i.e. the ratio of the dynamic to static value, is normally reported as function of strain rate. Under high strain rates, both the yield and ultimate stresses of reinforcing bars increase. A review of loading rate effects on concrete and reinforcing steel indicates that the modulus of elasticity and ultimate strain remain nearly constant, but that other bar properties, such as yield stress and strain, increase with rate. The ultimate stress increase is less significant (up to 5% at high strain rates, according to Keenan). Most of the available data regarding the yield stress DIF refers to the upper yield stress. Although the upper yield stress may appear less important than the lower yield stress under quasi static loading under dynamic loading the lower yield plateau shortens and the upper yield stress appears more significant. As indicated previously, the modulus of elasticity is usually found to remain constant under dynamic loading, hence the yield strain would increase with the yield stress DIF. The ultimate strain (at peak stress) remains constant and the static values indicated earlier apply.

The adopted DIF formulation was, for both yield and ultimate stress

$$DIF = \left(\frac{\dot{\varepsilon}}{10^{-4}}\right)^{\alpha} \qquad Eq-4.4$$

Where for the yield stress, $\alpha=\alpha_{fy}$ was found to be:

$$\alpha_{fy} = 0.074 - 0.040\frac{f_y}{60} \qquad Eq-4.5$$

and for the ultimate stress, $\alpha = \alpha_{fu}$ was found to be:

$$\alpha_{fu} = 0.019 - 0.009\frac{f_y}{60} \qquad Eq-4.6$$

and where the strain rate _H is in s-1 (1/second), and gy is the bar yield strength in ksi (if gy is in MPa, the 60 ksi denominator should be replaced by 414 MPa). Note that in both cases D is a function of gy. This formulation is valid for bars with yield stresses between 42 and 103 ksi (290 and 710 MPa) and for strain rates between 10-4 and 225 s-1.

A literature review was conducted to determine static and dynamic characteristics of steel reinforcing bars. It was found that the dynamic increase factor (DIF) for both yield and ultimate stress is inversely related to the yield stress itself. A formulation was proposed to determine the DIF as a function of strain rate and yield stress which appears to fit all the data properly. This formulation is valid for yield stresses between 42 and 103 ksi (290 and 710 MPa) and for strain rates between 10-4 and 225 s-1. This formulation gives the DIF for both yield and ultimate stress [4]

4.8- Ductility.

The basic reason most buildings do not collapse under seismic action is a material property called ductility. It is the property of a material to deform permanently without losing its strength, i.e. without decreasing its ability to resist during deformation. A piece of wire, e.g. an office clip bends but not brake. Due to ductility a system "resists" mobilizing all its reserves. [5]

The ductility capacity of reinforced concrete sections is usually expressed in terms of the curvature ductility ratio ($\mu_ø = ø_u/ø_y$) where $ø_y$ is the curvature of the section at first yield of the tensile reinforcement and $ø_u$ is the maximum curvature corresponding to a specific ultimate concrete compression strain. [2]

Azizinamini et al (1992) conducted full scale testing of columns with different transverse reinforcement details and found that the flexural capacity of columns increased with axial load but ductility reduced substantially. Test results indicate that increase in the amount of transverse reinforcement improves the ductility.[12]

4.9- Stress-Strain Relationships for Concrete and Steel.

Nevertheless, square confining steel does produce a significant increase in ductility, and some enhancement of strength has been observed by many investigators. It is evident from that confinement by transverse reinforcement has little effect on the stress-strain curve until the uniaxial strength of the concrete is approached. The shape of the stress-strain curve at high strains is a function of many variables, the major ones being the following.

1. The ratio of the volume of transverse steel to the volume of the concrete core, because a high transverse steel content will mean a high transverse confining pressure.
2. The yield strength of the transverse steel, because this gives an upper limit to the confining pressure.
3. The ratio of the spacing of the transverse steel to the dimensions of the concrete core, because a smaller spacing leads to more effective confinement, as illustrated in Fig. 4.11. The concrete is confined by arching of the concrete between the transverse bars and if the spacing is large it is evident that a large volume of the concrete cannot be confined and may spall away.
4. The ratio of the diameter of the transverse bar to the unsupported length of transverse bars in the case of rectangular stirrups or hoops, because a larger bar diameter leads to more effective confinement. Transverse bars of small diameter will act merely as ties between the corners

because the flexural stiffness of the hoop bar is small and the hoops bow outward rather than effectively confining the concrete in the regions between the corners. With a larger transverse bar diameter to unsupported length ratio, the area of concrete effectively confined will be larger because of the greater flexural stiffness of the hoop side. In the case of a circular spiral this variable has no significance: given its shape, the

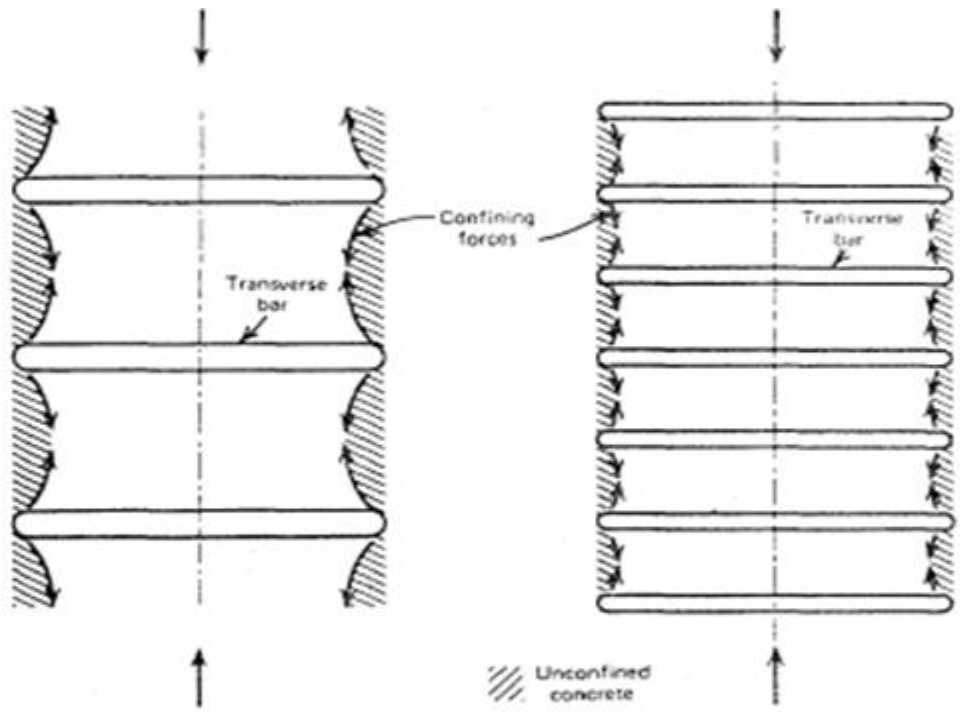

Figure 4-11: Effect of spacing of transverse steel on efficiency of confinement.

spiral will be in axial tension and will apply a uniform radial pressure to the concrete.

5. The content and size of longitudinal reinforcement, because this steel will also confine the concrete. Longitudinal bars are usually of large diameter, and the ratio of bar diameter to unsupported length is generally such that the bars can effectively confine the concrete. However, the longitudinal bars must be placed tightly against the transverse steel because the transverse steel provides the confining reactions to the longitudinal bars, and if movement of the longitudinal bars is necessary to bring them into effective contact with the transverse steel, the efficiency of the confinement will be reduced.

6. The strength of the concrete, because low-strength concrete is rather more ductile than high-strength concrete.

7. The rate of loading, because the stress-strain characteristics of concrete are time dependent.

.[10]

5-Methodology.

1-Searching, collecting and reading published materials (literatures, Journals, thesis, books etc.) related with the subject matter and deciding the model, the analysis and design approach.

- ✓ I have collected and read different related materials

2-Searching different FE software's and selecting the appropriate software which are suitable to design and analysis our model.

- ✓ I have selected sap2000 v.14 to be used as analysis and design software's because CSI FE software's are familiar in design of RC structures in addition to Microsoft excel and word 2007.

3-Modeling the representative 3D frame to match with our case axially loaded rectangular columns.

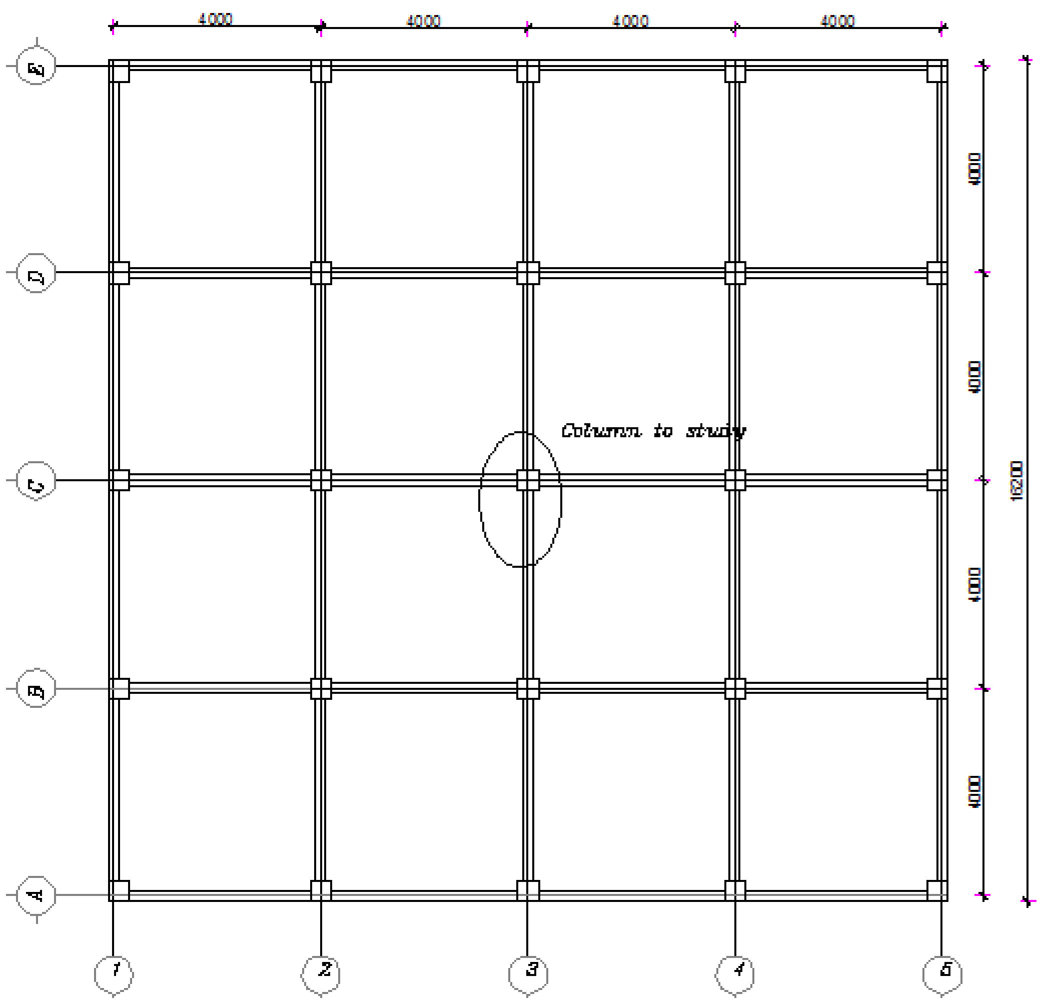

Figure-6-1:-Typical floor beam layout

- ✓ Purpose of the building is office for general use.
- ✓ Column at Axis-3 & Row-C will be designed to support 3, 6 & 9 stories.
- ✓ The staircase and lift shafts are assumed to be constructed and act separately.
- ✓ Concerned with only axially loaded rectangular RC columns I considered the gravity load case to analysis and design the multi-story frame.
- ✓ The frame is 11 story frame with 4 number of bays in x and y direction.
- ✓ The floor to floor story height is 3m and
- ✓ 4m grid spacing in x and y direction
- ✓ 2.5cm Concrete cover provided for columns and beams and 1.5cm for slab.
- ✓ Slab thickness is 15cm and beam dimension 30cm depth and 20cm width
- ✓ Rectangular column sections are assigned according to the gravity load they support using preliminary design.
- ✓ The case studied with C-25 and C-55 material grade for concrete (specified compressive strength) and S-300 and S-420 for rebar steel (yield strength).

Table-5-1:-Concrete material mechanical property

material	Compressive strength 28day.(Mpa)	Modulus of elasticity (Gpa)	Poisson's ratio	Unit weight (Kn/m2)
Concrete 1	25	30	0.2	25
Concrete 2	55	36	0.2	25

Table-5-2:-Steel material mechanical property

Material	Yield strength (Mpa)	Modulus of elasticity (Gpa)	Poisons ratio	Unit weight (Kn/m2)
Steel 1	420	200	0.3	7850
Steel 2	300	200	0.3	7850

Table-5-3:-Material models combination for concrete and steel

Material model	Concrete	Steel		
MM1	Concrete 1	Steel 1		
MM2	Concrete 1	Steel 2		
MM3	Concrete 2	Steel 1		
MM4	Concrete 2	Steel 2		

- ✓ **The load cases are**

For general use office use BS EN 1991-1-1 allows 2.5Kn/m2 imposed load , 1.5Kn/m2 for partition load and 1.25 Kn/m2 for finishes and services.

Permanent load g_k =self-weight + finish and services weigh t= 0.15 x 25 + 1.25 = 5kn/m2 and

Variable load q_k = imposed load + partition load = 2.5 + 1.5 = 4kn/m2

- ✓ **Load combination**

Combination 1 = $1.35g_k + 1.5q_k$ Ultimate limit state (gravity load case)

Combination 2 = $g_k + q_k + A_d$ Accidental load

- ✓ **Load transfer from slab to beams is in a trapezoidal manner.**

5-Perform analysis and design of the frame elements for the gravity load case only.

- ✓ After analysis of the structure the columns designed for the appropriate *cross section, longitudinal and transverse reinforcement (Table 3-4, 3-5 & 3-6)*. But the transverse reinforcement for the axially loaded columns provided with minimum reinforcement according to the British code.
- ✓ Checking the sections and reinforcement.

6-Taking sample columns at the center supporting (3, 6, & 9) stories to be studied for the lateral vehicular impact case.

- ✓ The columns at the middle of the floor (axis-3 & row-c) which supports *three, six and nine* story are our samples with pure axial load case. The sample columns redesigned with different cross section from the minimum 25cm with increment of 5cm without exceeding the minimum and maximum steel ratio 0.4% and 6% respectively specified in British code and with different(C-25 & C-55) concrete strength each with different steel yield strength(S-300 & S-420) *(Table 3-4, 3-5 & 3-6)*.
- ✓ But the transverse reinforcement for the axially loaded columns provided with minimum reinforcement according to the British code.

Table-5-4:-Material model 1

Column sample	No of stories above.	Column depth in (mm.)	Column width in (mm.)	Ultimate axial load in (KN.)	Longitudinal reinforcement	Minimum lateral reinforcement
1	9	400mm	400mm	2312.86	12 ϕ20 bar	ϕ6 c/c 240
2	9	450	450	2312.86	8 ϕ 20 bar	ϕ6 c/c 240
3	9	500	500	2312.86	4 ϕ 20 bar	ϕ6 c/c 240
4	6	300	300	1610.12	12 ϕ 20 bar	ϕ6 c/c 240
5	6	350	350	1610.12	12 ϕ 16 bar	ϕ6 c/c 190
6	6	400	400	1610.12	4 ϕ 16 bar	ϕ6 c/c 190
7	3	250	250	912.52	8 ϕ 16 bar	ϕ6 c/c 190

8	3	300	300	912.52	4 ϕ 14 bar	ϕ6 c/c 165

Table-5-5:-Material model 2

Column sample	No of stories above.	Column depth in (mm.)	Column width in (mm.)	Ultimate axial load in (KN.)	Longitudinal reinforcement	Minimum lateral reinforcement
1	9	450	450	2312.86	12 ϕ 20 bar	ϕ6 c/c 240
2	9	400	400	2312.86	16 ϕ 20 bar	ϕ6 c/c 240
3	6	400	400	1610.12	8 ϕ 14 bar	ϕ6 c/c 165
4	6	350	350	1610.12	12 ϕ 20 bar	ϕ6 c/c 240
5	6	300	300	1610.12	16 ϕ 20 bar	ϕ6 c/c 240
6	3	300	300	912.52	8 ϕ 12 bar	ϕ6 c/c 140
7	3	250	250	912.52	8 ϕ 20 bar	ϕ6 c/c 240

Table-5-6:-Material model 3

Column sample	No of stories above.	Column depth in (mm.)	Column width in (mm.)	Ultimate axial load in (KN.)	Longitudinal reinforcement	Minimum lateral reinforcement
1	9	300	300	2312.86	12 ϕ 16 bar	ϕ6 c/c 190
2	6	250	250	1610.12	8 ϕ 16 bar	ϕ6 c/c 190
3	3	200	200	912.52	4 ϕ 14 bar	ϕ6 c/c 140

Table-5-7:-Material model 4

Column sample	No of stories above.	Column depth in (mm.)	Column width in (mm.)	Ultimate axial load in (KN.)	Longitudinal reinforcement	Minimum lateral reinforcement
1	9	300	300	2312.86	12 ϕ 20 bar	ϕ6 c/c 240
2	6	250	250	1610.12	12 ϕ 16 bar	ϕ6 c/c 190
3	3	200	200	912.52	4 ϕ 16 bar	ϕ6 c/c 190

7-Selecting the appropriate equivalent lateral load magnitude from the code and also the impact height.

Table-5-8:-Indicative equivalent static design forces due to impact on superstructures.

Category of traffic	**Equivalent static design force F_{dx} [KN]**
Motorways and country national and main roads	500
Country roads in rural area	375
Roads in urban area	250
Cortyards and parking garages	75

(Source-EN 1991-1-7:2006(E))

- ✓ The impact height in our case taken as 0.75m above carriageway (decided after on site measurement).

8-Check the design for accidental load case for each sample column property.

- ✓ The sample columns (at axis-3 & row-c supporting 3, 6 & 9 stories) provided for gravity load case only will be checked for the accidental load case.
- ✓ The minimum lateral reinforcement will be checked since the shear resistance V_{rd} of a section shall not be less than the applied shear force V_{sd}.
- ✓ Shear resistance of members subjected to significant axial compression

$$V_c = 0.25 f_{cd} k_1 k_2 b_w d + 0.1 d \, {}^{b_w}/_{A_c} \, N_{sd}$$

9-Collect the result and interpret it.

- ✓ The accidental load case result output of the sample columns different probable sections will be compared with the ultimate limit state result of the gravity load case.

6-Result and discussion

6.1-Assemption and design codes used.

According to the British code BS 8110 part 1 clause 3.8.1 column means the greatest cross sectional dimension does not exceed four times the smallest dimension.

Columns classified as short columns when the ratios l_{ex}/h and l_{ey}/b are both less than 15 for braced and 10 for unbraced. And the column is slender when the ratios are larger than the values given above.

$$l_{ex} = column\ effective\ length\ in\ x\ direction$$

$$l_{ey} = column\ effective\ length\ in\ y\ direction$$

In our case the sear capacity of sections calculated according to EBCS 96 the analysis and design of sections carried on according to the British code of BS 8110 and the allowable sections was also proportioned to carry the design ultimate axial load using the formula in clause 3.8.4.4 for short braced columns supporting an approximately symmetrical arrangement of beams.

These beams must be designed for uniformly distributed imposed load and the span must not differ by more than 15% of the longer span and the ultimate load is given by the expression.

$$P_u = 0.35 f_{cu} A_c + 0.67 A_{sc} f_y \qquad eq - 6.1$$

$$f_{cu} = Cube\ strength\ of\ concrete$$

$$A_c = Area\ of\ concrete\ section$$

$$A_{sc} = Area\ of\ compression\ steel$$

$$f_y = Yeild\ strength\ of\ reinforcement\ steel$$

The lateral force assumed to be supported by independent structure we design the allowable section according to British code requirement clause 3.8.4.4 for short braced axially loaded columns eccentricity due to loading is considered by reducing the capacity for axial load by 10%.

Table 6.1- allowable section designing for MM1

9 storis above	***Fcu***	**25**	**Mpa**	
	fy	**420**	**Mpa**	
	Pu	2,312.86	2,312.86	2,312.86
	H	400	450	500
	Ag	160,000.00	202,500.00	250,000.00

	As	0.0033481	0.00198417	0.00045978
		3,348.10	1,984.17	459.78
	Rho	2.0926	0.9798	0.1839
	Bar size	***20***	***20***	***20***
	No of bar calc	10.6627	6.3190	1.4643
	No of bar	***12***	***8***	***4***
	As prov	3,768.00	2,512.00	1,256.00
	Rho prov	2.3550	1.2405	0.5024
6 storis above	***Fcu***	**25**	**Mpa**	
	fy	**420**	**Mpa**	
	Pu	1,610.21	1,610.21	1,610.21
	H	300	350	400
	Ag	90,000.00	122,500.00	160,000.00
	As	0.00301747	0.00197447	0.000771
		3,017.47	1,974.47	771.00
	Rho	3.35274772	1.61181151	0.48187695
	Bar size	***20***	***16***	***16***
	No of bar calc	9.60978647	9.82518461	3.83659991
	No of bar	***12***	***12***	***4***
	As prov	3,768.00	2,411.52	803.84
	Rho prov	4.1867	1.9686	0.5024
3 storis above	***Fcu***	**25**	**Mpa**	
	fy	**420**	**Mpa**	
	Pu	912.52	912.52	
	H	250	300	
	Ag	62,500.00	90,000.00	
	As	0.0013411	0.00045854	
		1,341.08	458.54	
	Rho	2.1457	0.5095	
	Bar size	***16***	***14***	
	No of bar calc	6.6734	2.9802	
	No of bar	***8***	***4***	
	As prov	1,607.68	615.44	
	Rho prov	2.5723	0.6838	

Table 6.2- allowable section designing for MM2

9 storis above	***Fcu***	**25**	**Mpa**	
	fy	**300**	**Mpa**	
	Pu	2,312.86	2,312.86	
	H	400	450	
	Ag	160,000.00	202,500.00	
	As	0.0047483	0.00281397	
		4,748.30	2,813.97	
	Rho	2.9677	1.3896	
	Bar size	***20***	***20***	
	No of bar calc	15.1220	8.9617	
	No of bar	***16***	***12***	
	As prov	5,024.00	3,768.00	
	Rho prov	3.1400	1.8607	
6 storis above	***Fcu***	**25**	**Mpa**	
	fy	**300**	**Mpa**	
	Pu	1,610.21	1,610.21	1,610.21
	H	300	350	400
	Ag	90,000.00	122,500.00	160,000.00
	As	0.0042794	0.0028002	0.00109344
		4,279.40	2,800.20	1,093.44
	Rho	4.75488513	2.28587989	0.68340052
	Bar size	***20***	***20***	***14***
	No of bar calc	13.6286517	8.91784351	7.1067258
	No of bar	***16***	***12***	***8***
	As prov	5,024.00	3,768.00	1,230.88
	Rho prov	5.5822	3.0759	0.7693
3 storis above	***Fcu***	**25**	**Mpa**	
	fy	**300**	**Mpa**	
	Pu	912.52	912.52	
	H	250	300	
	Ag	62,500.00	90,000.00	

	As	0.0019019	0.0006503
		1,901.92	650.30
	Rho	3.0431	0.7226
	Bar size	***20***	***12***
	No of bar calc	6.0571	5.7528
	No of bar	***8***	***8***
	As prov	2,512.00	904.32
	Rho prov	4.0192	1.0048

Table 6.3- allowable section designed with MM3

9 storis above	***Fcu***	**55**	**Mpa**
	fy	**420**	**Mpa**
	Pu	2,312.86	
	H	300	
	Ag	90,000.00	
	As	0.00221385	
		2,213.85	
	Rho	2.4598	
	Bar size	***16***	
	No of bar calc	11.0164	
	No of bar	***12***	
	As prov	2,411.52	
	Rho prov	2.67946667	
6 storis above	***Fcu***	**55**	**Mpa**
	fy	**420**	**Mpa**
	Pu	1,610.21	
	H	250	
	Ag	62,500.00	
	As	0.0015529	
		1,552.89	
	Rho	2.4846172	
	Bar size	***16***	
	No of bar	7.7273375	

	calc		
	No of bar	***8***	
	As prov	1,607.68	
	Rho prov	2.572288	
3 storis above	***Fcu***	**55**	**Mpa**
	fy	**420**	**Mpa**
	Pu	912.52	
	H	200	
	Ag	40,000.00	
	As	0.0005437	
		543.66	
	Rho	1.3591	
	Bar size	***14***	
	No of bar calc	3.5335	
	No of bar	***4***	
	As prov	615.44	
	Rho prov	1.5386	

Table 6.4- allowable section designing for MM4

9 storis above	***Fcu***	**55**	**Mpa**
	fy	**300**	**Mpa**
	Pu	2,312.86	
	H	300	
	Ag	90,000.00	
	As	0.0031932	
		3,193.18	
	Rho	3.5480	
	Bar size	***20***	
	No of bar calc	10.1694	
	No of bar	***12***	
	As prov	3,768.00	
	Rho prov	4.1866667	

6 storis above	***Fcu***	**55**	**Mpa**
	fy	**300**	**Mpa**
	Pu	1,610.21	
	H	250	
	Ag	62,500.00	
	As	0.0022398	
		2,239.83	
	Rho	3.5837271	
	Bar size	***16***	
	No of bar calc	11.145648	
	No of bar	***12***	
	As prov	2,411.52	
	Rho prov	3.858432	
3 storis above	***Fcu***	**55**	**Mpa**
	fy	**300**	**Mpa**
	Pu	912.52	
	H	200	
	Ag	40,000.00	
	As	0.0007842	
		784.15	
	Rho	1.9604	
	Bar size	***16***	
	No of bar calc	3.9020	
	No of bar	***4***	
	As prov	803.84	
	Rho prov	2.0096	

The equivalent static design force for impact is according to European code EN 1991-1-7:2006(E) table 6.2 in the category of courtyards and parking garages which is 75KN in the direction of normal travel.

Slender sections do not included in our case to exclude the appearance of 2nd order moments since we are to consider gravitational load only.

<u>PMM ratio or capacity ratio (*CR*):-</u> is an indication of the stress condition of the column with respect to the capacity of the column.

• If CR=1 the point lies on the interaction surface and the column is *stressed to capacity*.
• If CR<1 the point lies within the interaction volume and the column capacity is *adequate*.
• If CR>1 the point lies outside the interaction volume and the column is *overstressed.*

It's also the ratio on the e_b/t line which is 0 at the origin and 1 at the line and the curve intersection point

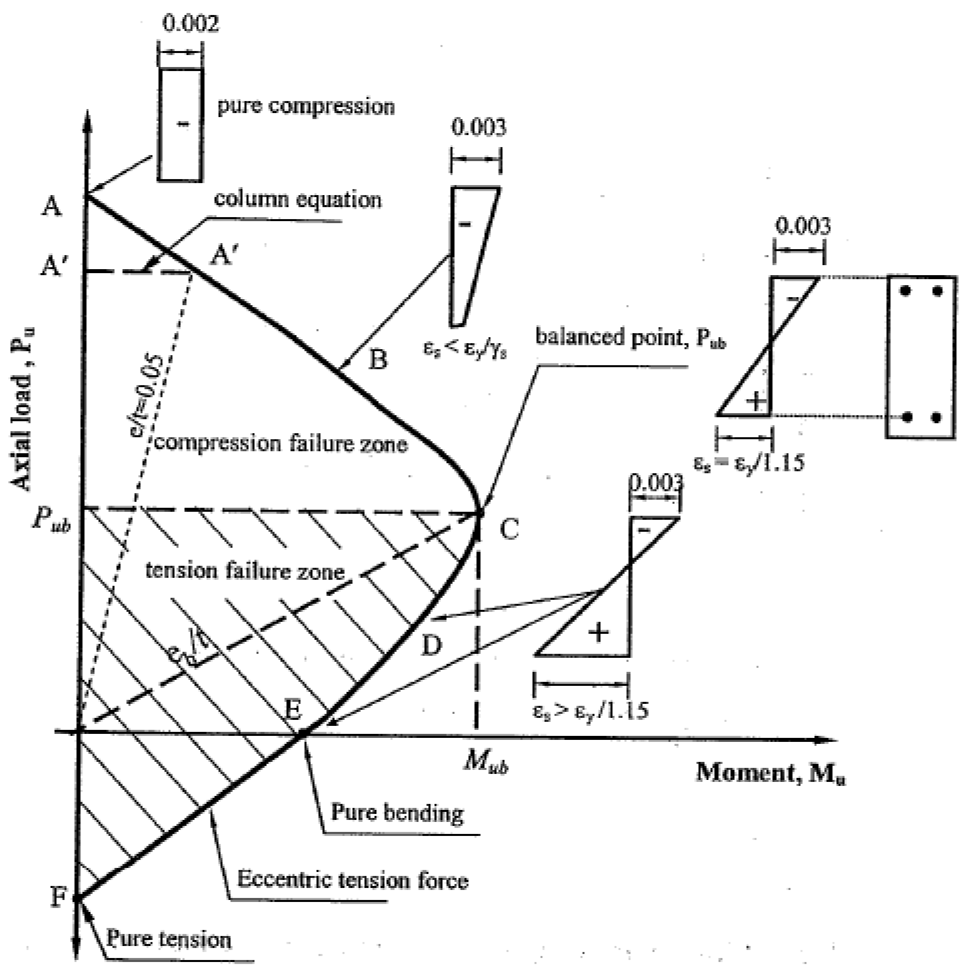

Figure 6.1: Models of failure for a section subjected to eccentric forces.

<u>Percentage difference (*PD*):-</u> is the percentage difference of the capacity ratio of the *gravity load case* (combination 1) with respect to *Impact load case* (combination 2)

$$PD = \left(\frac{CR_{comb\ 1} - CR_{comb\ 2}}{CR_{comb\ 1}}\right) \times 100\%$$

• If PD is negative the section is more stressed in Impact load case.

• If PD is positive the section is more stressed in Gravity load case.

N.B:- Annex-3 is graphical presentation of Annex-1.

6.2-Effect of using different grade of concrete and steel on PMM ratio for ultimate (gravity) & accidental load case.

The analysis result shown in annex 1 and 3 for the gravity load case shows the most stressed section is at mid height of the column with the least at the top. But for impact case the more stressed section is from the bottom up to the impact level 0.75m above. But the section above the impact level up to the top is less stressed.

Only in the case of 25x25-8-16 MM1 and 25x25-8-20 MM2 the capacity ratio of Impact load case exceeds the gravity load case on sections from bottom up to the impact level and we see negative PD (*percentage difference*) but the column is safe in both cases not exceeding their capacity.

For MM3 and MM4 the 20cm width columns fail due to buckling before they reach their maximum axial load carrying capacity because the slenderness ratio of the column exceeds the limit set by the British code.

For all columns with different material model the least PD appears on sections down from the impact level that means these sections are more sensitive to impact compared with sections above.

According to annex 1 and 3 the column sections with lesser steel ratio (higher concrete section) will be safer for Impact load case compared with higher steel ratio (lesser concrete section) even if they are designed to carry the same axial load (we can see it in all cases in annex -3 graphically).

Table 6.5- effect of using different concrete and steel grade at column bottom section.

Column section (mm^2)	Rebar	Stories above	Axial load (KN.)	Concrete grade (Mpa)	Steel grade (Mpa)	Steel ratio	Capacity ratio at bottom of column.		Percentage difference
							Comb 1	Comb 2	
25x25	8-dia16	3	671.49	25	420	2.57%	0.814	0.927	-13.80
25x25	8-dia20	3	671.49	25	300	4.02%	0.746	0.834	-11.80
20x20	4-dia14	3	622.00	55	420	1.54%	0.823	1.223	-32.74
20x20	4-dia16	3	671.49	55	300	2.00%	0.835	1.207	-30.54
30x30	12-dia20	6	1185.55	25	420	4.19%	0.762	0.655	13.98
30x30	16-dia20	6	1165.11	25	300	5.58%	0.767	0.651	15.21
25x25	8-dia16	6	1102.00	55	420	2.57%	0.830	0.802	3.54
30x30	12-dia16	6	1110.25	55	300	3.86%	0.814	0.781	4.21
40x40	12-dia20	9	1186.77	25	420	2.36%	0.819	0.616	24.76
40x40	16-dia20	9	1681.51	25	300	3.14%	0.839	0.622	25.89
30x30	12-dia16	9	1584.00	55	420	2.68%	0.817	0.677	20.55
30x30	12-dia20	9	1584.34	55	300	4.19%	0.792	0.650	21.78

As we can see from annex1 and 3 when the number of story's increases the PD also increases that means as number of story to support increases the susceptibility to failure due to Impact decreases because the sections for gravity case is higher relative to columns to support lesser story .

For column section from bottom to top and for allowable sections from lower to higher which support the same axial load the percentage difference increases as we can see in annex 3.

For the same column section with different steel grade (reinforcement ratio) to support the same axial load the percentage difference is almost the same which shows that it doesn't depend on the steel grade.

The allowable sections for higher and ordinary strength concrete doesn't fit because the section which is for one case over or under reinforced for the other case that shows column to support lesser stories using high strength concrete limits our choice of economical sections to be used.

As we can see from the sap output result in *annex 1and 3* for columns supporting stories not more than four using high strength concrete makes the section lesser and the stress ratio for impact relative to the gravity one will be lower compared with ordinary concrete strength .

In annex 3 we can see that column with materials property of MM3 and MM4 the capacity ratio for accidental case is higher and more stressed relative to the gravity load case even if they are carrying the same axial load.

Ordinary strength concrete is preferable to resist Impact because we can get high sections relatively. As we can see in the annex 3 columns sections with lower longitudinal reinforcement ratio will be safer that those with higher ratio.

As shown in table 6.1, 6.2, 6.3 & 6.4 in higher strength concrete columns the allowable sections are limited to only one for each axial load case.

The higher strength columns sections with different steel grade and the same axial load the percentage difference is the same irrespective of steel ratio.

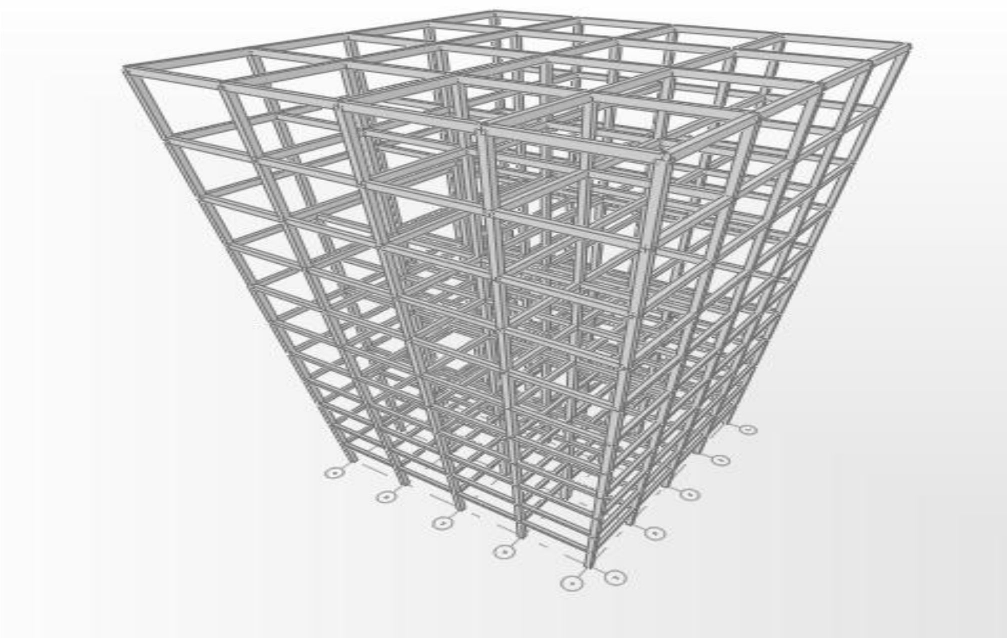

Figure 6.2: 3D structural view of the model building

As shown in annex 1 and 3 the percentage difference for higher strength concrete from bottom to top of the column and from lesser to higher sections increases considerably that means the critical section is from impact level to the bottom.

6.3-Shear resistance of section

The column impact resistance is a lot more than the applied shear (impact) as shown in annex 2 because the applied compression force on the cross section will increase the area in compression and thus enhance the shear capacity.

According to EBCS code the shear resistance of the concrete section is

$$V_c = 0.25 f_{cd} k_1 k_2 b_w d + 0.1 d\, {}^{b_w}/_{A_c}\, N_{sd} \qquad eq - 6.2$$

$$k_1 = (1 + 50\rho) \leq 2.0$$

$$k_2 = 1.6 - d \geq 1.0$$

$$\rho = A_s / b_w d$$

As we can see the shear strength of the section depends on concrete grade, cross section, steel ratio and the applied axial load the shear capacity of the probable sections is analyzed using eq-6.2 in the preceding table..

According to tables in all cases the shear resistance of the section is higher than the ultimate applied shear.

As shown in annex 2 the maximum moment and shear is on sections below the impact level and the minimum above the impact level and that's why the capacity ratio (PMM ratio) for accidental case increases due to the additional moment from the impact.

The output shows the shear reinforcement for all cases is the minimum according to BS 8110 part, clause 3.4.5.2

$$A_{sv} = 0.4 b_v s_v / 0.87 f_{yv} \qquad eq - 6.3$$

$$b_v = width\ of\ the\ column\ section$$

$$s_v = spacng\ of\ transverse\ reinforcement$$

$$f_{yv} = Yeild\ strength\ of\ transverse\ reinforcement$$

As we can see the transverse reinforcement in the output for all material models is the minimum area reinforcement required according to British code.

As annex 2 and table 6.1, 6.2, 6.3 & 6.4 for columns supporting 3 stories with whatever concrete grade the shear strength of the section could have been exceeded if it is not for the axial load contribution for the section shear resistance capacity.

Table 6.6- shear capacity of sections for MM1.

description	**unit**	**1-Number of stories supported by the column**							
		3	**3**	**6**	**6**	**6**	**9**	**9**	**9**
C	Mpa	25	25	25	25	25	25	25	25
f_{ck}	Mpa	20	20	20	20	20	20	20	20
f_{ctm}	Mpa	2.21	2.21	2.21	2.21	2.21	2.21	2.21	2.21
f_{ctk}	Mpa	1.55	1.55	1.55	1.55	1.55	1.55	1.55	1.55
f_{ctd}	Mpa	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03
N_{sd}	KN	661.61	671.50	1,148.86	1,165.11	1,185.55	1,658.82	1,673.48	1,686.77
Dia d_b	mm	16	14	20	16	16	20	20	20
No. of d_b		8	4	12	12	4	12	8	4
A_s	mm^2	1,607.68	615.44	3,768.00	2,411.52	803.84	3,768.00	2,512.00	1,256.00

A_c	mm^2	60,892.32	89,384.56	86,232.00	120,088.48	159,196.16	156,232.00	199,988.00	248,744.00
D	mm	250.00	300.00	300.00	350.00	400.00	400.00	450.00	500.00
b_w	mm	250.00	300.00	300.00	350.00	400.00	400.00	450.00	500.00
d	mm	211.00	262.00	259.00	311.00	361.00	359.00	409.00	459.00
Rebar		0.0305	0.0078	0.0485	0.0222	0.0056	0.0262	0.0136	0.0055
rho		3.05%	0.78%	4.85%	2.22%	0.56%	2.62%	1.36%	0.55%
k_1		2.000	1.392	2.000	2.000	1.278	2.000	1.682	1.274
k_2		1.39	1.34	1.34	1.29	1.24	1.24	1.19	1.14
V_{cn}	KN	57.31	59.05	103.52	105.61	107.54	152.47	154.01	155.63
V_c	KN	**95.10**	**96.79**	**157.26**	**177.97**	**166.52**	**244.38**	**249.12**	**241.63**

Table 6.7- shear capacity of sections for MM2

description	**unit**	**2-Number of stories supported by the column**						
		3	**3**	**6**	**6**	**6**	**9**	**9**
C	Mpa	25	25	25	25	25	25	25
f_{ck}	Mpa	20	20	20	20	20	20	20
f_{ctm}	Mpa	2.21	2.21	2.21	2.21	2.21	2.21	2.21
f_{ctk}	Mpa	1.55	1.55	1.55	1.55	1.55	1.55	1.55
f_{ctd}	Mpa	1.03	1.03	1.03	1.03	1.03	1.03	1.03
N_{sd}	KN	661.61	671.50	1,148.86	1,165.11	1,185.55	1,658.82	1,673.48
Dia d_b	mm	20	8	20	20	14	20	20
No. of d_b		8	12	16	12	8	16	12
A_s	mm^2	2,512.00	602.88	5,024.00	3,768.00	1,230.88	5,024.00	3,768.00
A_c	mm^2	59,988.00	89,397.12	84,976.00	118,732.00	158,769.12	154,976.00	198,732.00
D	mm	250.00	300.00	300.00	350.00	400.00	400.00	450.00
b_w	mm	250.00	300.00	300.00	350.00	400.00	400.00	450.00
d	mm	209.00	265.00	259.00	309.00	362.00	359.00	409.00

Rebar		0.0481	0.0076	0.0647	0.0348	0.0085	0.0350	0.0205
rho		4.81%	0.76%	6.47%	3.48%	0.85%	3.50%	2.05%
k_1		2.000	1.379	2.000	2.000	1.425	2.000	2.000
k_2		1.39	1.34	1.34	1.29	1.24	1.24	1.19
V_{cn}	KN	57.63	59.72	105.05	106.13	108.12	153.71	154.98
V_c	KN	**95.11**	**97.46**	**158.79**	**178.14**	**174.00**	**245.62**	**268.04**

Table 6.8- shear capacity of sections for MM3

description	**unit**	**3-Number of stories supported by the column**		
		3	**6**	**9**
C	Mpa	55	55	55
f_{ck}	Mpa	44	44	44
f_{ctm}	Mpa	3.74	3.74	3.74
f_{ctk}	Mpa	2.62	2.62	2.62
f_{ctd}	Mpa	1.74	1.74	1.74
N_{sd}	KN	622.00	1,102.00	1,584.40
Dia d_b	mm	14	16	16
No. of d_b		4	8	12
A_s	mm^2	615.44	1,607.68	2,411.52
A_c	mm^2	39,384.56	60,892.32	87,588.48
D	mm	200.00	250.00	300.00
b_w	mm	200.00	250.00	300.00
d	mm	162.00	211.00	261.00
Rebar		0.0190	0.0305	0.0308
rho		1.90%	3.05%	3.08%
k_1		1.950	2.000	2.000

k_2		1.44	1.39	1.34
V_{cn}	KN	51.17	95.46	141.64
V_c	KN	**90.80**	**159.39**	**233.11**

Table 6.9- shear capacity of sections for MM4

description	**unit**	**4-Number of stories supported by the column**		
		3	**6**	**9**
C	Mpa	55	55	55
f_{ck}	Mpa	44	44	44
f_{ctm}	Mpa	3.74	3.74	3.74
f_{ctk}	Mpa	2.62	2.62	2.62
f_{ctd}	Mpa	1.74	1.74	1.74
N_{sd}	KN	622.00	1,110.25	1,584.40
Dia d_b	mm	16	16	20
No. of d_b		4	12	12
A_s	mm^2	803.84	2,411.52	3,768.00
A_c	mm^2	39,196.16	60,088.48	86,232.00
D	mm	200.00	250.00	300.00
b_w	mm	200.00	250.00	300.00
d	mm	161.00	211.00	259.00
Rebar		0.0250	0.0457	0.0485
rho		2.50%	4.57%	4.85%
k_1		2.000	2.000	2.000
k_2		1.44	1.39	1.34
V_{cn}	KN	51.10	97.47	142.76
V_c	KN	**91.52**	**161.39**	**233.67**

7-Conclusion

As to our research findings RC columns conventionally designed for axial load (gravity load case) only their resistance for impact increases as axial load and area of concrete increases.

Even if RC columns designed only for axial load due to gravity load case when stories above increase the axial load being higher and higher and the fear of failure due to accidental impact decreases.

For the same column section with different steel grade (reinforcement ratio) to support the same axial load the percentage difference is almost the same which shows that it doesn’t depend on the steel grade.

Columns with higher concrete strength has higher capacity ratio for accidental load case when compared with ordinary concrete strength columns designed to carry the same axial load because they are shear sensitive.

Columns sections shear capacity increases with axial load.

For higher and lower grade RC columns the conventional designed minimum transverse reinforcements are enough for the higher shear induced at shear critical region from bottom of column to the impact level.

Column sections with lower steel ratio preferable for columns susceptible to vehicular impact since they give higher concrete sections when they designed for axial load and the shear capacity also increases proportionally with the concrete section.

Using high strength grade concrete gives less alternative economical concrete section to carry the axial load.

Generally all columns designed with different material model from MM1 up to MM4 have enough capacity for flexure, axial and shear induced from accidental impacts even they are designed for axial load only.

8- Recommendation

This research looks at only columns designed for axial load only due to gravity load case and for symmetric rectangular shape floor area which is the same for all stories. Taking this only one of the design features it's better to future researches to look on columns designed for uniaxial and biaxial load cases for buildings designed to lateral load case in addition to the gravity load.

Also it's recommended to make researches for buildings which are nonsymmetrical horizontally and vertically and with different material models.

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ANNEX

10-Annex-1

Sap2000 V.14 PMM Output Table for Different material models.

TABLE: Concrete Details for Sap2000 output for material model 1 - Column Summary Data - BS8110 97

Frame	DesignSect	DesignType	Location	PMMCombo	PMMRatio	VMajCombo	VMajRebar	VMinCombo	VMinRebar
Text	Text	Text	m	Text	Unitless	Text	m2/m	Text	m2/m
676	25x25 8-16 -(3)	Column	0	COMB1	0.814154	COMB1	0.00025	COMB1	0.00025
676	25x25 8-16 -(3)	Column	0.75	COMB1	0.8735	COMB1	0.00025	COMB1	0.00025
676	25x25 8-16 -(3)	Column	0.75	COMB1	0.8735	COMB1	0.00025	COMB1	0.00025
676	25x25 8-16 -(3)	Column	1.5	COMB1	0.933575	COMB1	0.00025	COMB1	0.00025
676	25x25 8-16 -(3)	Column	3	COMB1	0.808872	COMB1	0.00025	COMB1	0.00025
676	25x25 8-16 -(3)	Column	0	COMB2	0.926873	COMB2	0.00025	COMB2	0.00025
676	25x25 8-16 -(3)	Column	0.75	COMB2	0.823327	COMB2	0.00025	COMB2	0.00025
676	25x25 8-16 -(3)	Column	0.75	COMB2	0.823327	COMB2	0.00025	COMB2	0.00025
676	25x25 8-16 -(3)	Column	1.5	COMB2	0.70947	COMB2	0.00025	COMB2	0.00025
676	25x25 8-16 -(3)	Column	3	COMB2	0.637617	COMB2	0.00025	COMB2	0.00025
676	30x30 4-14 -(3)	Column	0	COMB1	0.859333	COMB1	0.0003	COMB1	0.0003
676	30x30 4-14 -(3)	Column	0.75	COMB1	0.875063	COMB1	0.0003	COMB1	0.0003
676	30x30 4-14 -(3)	Column	0.75	COMB1	0.875063	COMB1	0.0003	COMB1	0.0003
676	30x30 4-14 -(3)	Column	1.5	COMB1	0.889724	COMB1	0.0003	COMB1	0.0003
676	30x30 4-14 -(3)	Column	3	COMB1	0.851421	COMB1	0.0003	COMB1	0.0003
676	30x30 4-14 -(3)	Column	0	COMB2	0.808909	COMB2	0.0003	COMB2	0.0003
676	30x30 4-14 -(3)	Column	0.75	COMB2	0.785174	COMB2	0.0003	COMB2	0.0003
676	30x30 4-14 -(3)	Column	0.75	COMB2	0.785174	COMB2	0.0003	COMB2	0.0003
676	30x30 4-14 -(3)	Column	1.5	COMB2	0.684789	COMB2	0.0003	COMB2	0.0003

676	30x30 4-14 -(3)	Column	3	COMB2	0.627081	COMB2	0.0003	COMB2	0.0003
673	30x30 12-20 (6)	Column	0	COMB1	0.761788	COMB1	0.0003	COMB1	0.0003
673	30x30 12-20 (6)	Column	0.75	COMB1	0.785716	COMB1	0.0003	COMB1	0.0003
673	30x30 12-20 (6)	Column	0.75	COMB1	0.785716	COMB1	0.0003	COMB1	0.0003
673	30x30 12-20 (6)	Column	1.5	COMB1	0.801003	COMB1	0.0003	COMB1	0.0003
673	30x30 12-20 (6)	Column	3	COMB1	0.753474	COMB1	0.0003	COMB1	0.0003
673	30x30 12-20 (6)	Column	0	COMB2	0.65526	COMB2	0.0003	COMB2	0.0003
673	30x30 12-20 (6)	Column	0.75	COMB2	0.644877	COMB2	0.0003	COMB2	0.0003
673	30x30 12-20 (6)	Column	0.75	COMB2	0.644877	COMB2	0.0003	COMB2	0.0003
673	30x30 12-20 (6)	Column	1.5	COMB2	0.587689	COMB2	0.0003	COMB2	0.0003
673	30x30 12-20 (6)	Column	3	COMB2	0.558065	COMB2	0.0003	COMB2	0.0003
673	35x35 12-16 -(6)	Column	0	COMB1	0.813343	COMB1	0.00035	COMB1	0.00035
673	35x35 12-16 -(6)	Column	0.75	COMB1	0.827847	COMB1	0.00035	COMB1	0.00035
673	35x35 12-16 -(6)	Column	0.75	COMB1	0.827847	COMB1	0.00035	COMB1	0.00035
673	35x35 12-16 -(6)	Column	1.5	COMB1	0.835835	COMB1	0.00035	COMB1	0.00035
673	35x35 12-16 -(6)	Column	3	COMB1	0.807466	COMB1	0.00035	COMB1	0.00035
673	35x35 12-16 -(6)	Column	0	COMB2	0.642995	COMB2	0.00035	COMB2	0.00035
673	35x35 12-16 -(6)	Column	0.75	COMB2	0.661599	COMB2	0.00035	COMB2	0.00035
673	35x35 12-16 -(6)	Column	0.75	COMB2	0.661599	COMB2	0.00035	COMB2	0.00035
673	35x35 12-16 -(6)	Column	1.5	COMB2	0.613269	COMB2	0.00035	COMB2	0.00035
673	35x35 12-16 -(6)	Column	3	COMB2	0.581464	COMB2	0.00035	COMB2	0.00035
673	40x40 4-16 -(6)	Column	0	COMB1	0.894554	COMB1	0.0004	COMB1	0.0004
673	40x40 4-16 -(6)	Column	0.75	COMB1	0.896849	COMB1	0.0004	COMB1	0.0004
673	40x40 4-16 -(6)	Column	0.75	COMB1	0.896849	COMB1	0.0004	COMB1	0.0004
673	40x40 4-16 -(6)	Column	1.5	COMB1	0.897426	COMB1	0.0004	COMB1	0.0004
673	40x40 4-16 -(6)	Column	3	COMB1	0.886254	COMB1	0.0004	COMB1	0.0004
673	40x40 4-16 -(6)	Column	0	COMB2	0.666741	COMB2	0.0004	COMB2	0.0004

673	40x40 4-16 -(6)	Column	0.75	COMB2	0.700603	COMB2	0.0004	COMB2	0.0004
673	40x40 4-16 -(6)	Column	0.75	COMB2	0.700603	COMB2	0.0004	COMB2	0.0004
673	40x40 4-16 -(6)	Column	1.5	COMB2	0.659455	COMB2	0.0004	COMB2	0.0004
673	40x40 4-16 -(6)	Column	3	COMB2	0.626956	COMB2	0.0004	COMB2	0.0004
670	40x40 12-20-(9)	Column	0	COMB1	0.819115	COMB1	0.0004	COMB1	0.0004
670	40x40 12-20-(9)	Column	0.75	COMB1	0.828804	COMB1	0.0004	COMB1	0.0004
670	40x40 12-20-(9)	Column	0.75	COMB1	0.828804	COMB1	0.0004	COMB1	0.0004
670	40x40 12-20-(9)	Column	1.5	COMB1	0.832004	COMB1	0.0004	COMB1	0.0004
670	40x40 12-20-(9)	Column	3	COMB1	0.813675	COMB1	0.0004	COMB1	0.0004
670	40x40 12-20-(9)	Column	0	COMB2	0.61627	COMB2	0.0004	COMB2	0.0004
670	40x40 12-20-(9)	Column	0.75	COMB2	0.621518	COMB2	0.0004	COMB2	0.0004
670	40x40 12-20-(9)	Column	0.75	COMB2	0.621518	COMB2	0.0004	COMB2	0.0004
670	40x40 12-20-(9)	Column	1.5	COMB2	0.597469	COMB2	0.0004	COMB2	0.0004
670	40x40 12-20-(9)	Column	3	COMB2	0.57957	COMB2	0.0004	COMB2	0.0004
670	45x45 8-20 -(9)	Column	0	COMB1	0.813926	COMB1	0.00045	COMB1	0.00045
670	45x45 8-20 -(9)	Column	0.75	COMB1	0.817663	COMB1	0.00045	COMB1	0.00045
670	45x45 8-20 -(9)	Column	0.75	COMB1	0.817663	COMB1	0.00045	COMB1	0.00045
670	45x45 8-20 -(9)	Column	1.5	COMB1	0.81984	COMB1	0.00045	COMB1	0.00045
670	45x45 8-20 -(9)	Column	3	COMB1	0.807152	COMB1	0.00045	COMB1	0.00045
670	45x45 8-20 -(9)	Column	0	COMB2	0.596848	COMB2	0.00045	COMB2	0.00045
670	45x45 8-20 -(9)	Column	0.75	COMB2	0.608811	COMB2	0.00045	COMB2	0.00045
670	45x45 8-20 -(9)	Column	0.75	COMB2	0.608811	COMB2	0.00045	COMB2	0.00045
670	45x45 8-20 -(9)	Column	1.5	COMB2	0.589091	COMB2	0.00045	COMB2	0.00045
670	45x45 8-20 -(9)	Column	3	COMB2	0.571531	COMB2	0.00045	COMB2	0.00045
670	50x50 4-20 -(9)	Column	0	COMB1	0.795824	COMB1	0.0005	COMB1	0.0005
670	50x50 4-20 -(9)	Column	0.75	COMB1	0.795343	COMB1	0.0005	COMB1	0.0005
670	50x50 4-20 -(9)	Column	0.75	COMB1	0.795343	COMB1	0.0005	COMB1	0.0005

670	50x50 4-20 -(9)	Column	1.5	COMB1	0.794116	COMB1	0.0005	COMB1	0.0005
670	50x50 4-20 -(9)	Column	3	COMB1	0.78771	COMB1	0.0005	COMB1	0.0005
670	50x50 4-20 -(9)	Column	0	COMB2	0.570736	COMB2	0.0005	COMB2	0.0005
670	50x50 4-20 -(9)	Column	0.75	COMB2	0.58642	COMB2	0.0005	COMB2	0.0005
670	50x50 4-20 -(9)	Column	0.75	COMB2	0.58642	COMB2	0.0005	COMB2	0.0005
670	50x50 4-20 -(9)	Column	1.5	COMB2	0.569133	COMB2	0.0005	COMB2	0.0005
670	50x50 4-20 -(9)	Column	3	COMB2	0.55571	COMB2	0.0005	COMB2	0.0005

TABLE: Concrete Details for Sap2000 output for material model 2 - Column Summary Data - BS8110 97									
Frame	**DesignSect**	**DesignType**	**Location**	**PMMCombo**	**PMMRatio**	**VMajCombo**	**VMajRebar**	**VMinCombo**	**VMinRebar**
Text	Text	Text	m	Text	Unitless	Text	m2/m	Text	m2/m
676	column 25x25 8-dia20(3)	Column	0	COMB1	0.746397	COMB1	0.00035	COMB1	0.00035
676	column 25x25 8-dia20(3)	Column	0.75	COMB1	0.785773	COMB1	0.00035	COMB1	0.00035
676	column 25x25 8-dia20(3)	Column	0.75	COMB1	0.785773	COMB1	0.00035	COMB1	0.00035
676	column 25x25 8-dia20(3)	Column	1.5	COMB1	0.836175	COMB1	0.00035	COMB1	0.00035
676	column 25x25 8-dia20(3)	Column	3	COMB1	0.741413	COMB1	0.00035	COMB1	0.00035
676	column 25x25 8-dia20(3)	Column	0	COMB2	0.834428	COMB2	0.00035	COMB2	0.00035
676	column 25x25 8-dia20(3)	Column	0.75	COMB2	0.743262	COMB2	0.00035	COMB2	0.00035
676	column 25x25 8-dia20(3)	Column	0.75	COMB2	0.743262	COMB2	0.00035	COMB2	0.00035
676	column 25x25 8-dia20(3)	Column	1.5	COMB2	0.638682	COMB2	0.00035	COMB2	0.00035
676	column 25x25 8-dia20(3)	Column	3	COMB2	0.581004	COMB2	0.00035	COMB2	0.00035
676	column 30x30 8-dia12(3)	Column	0	COMB1	0.826276	COMB1	0.00042	COMB1	0.00042
676	column 30x30 8-dia12(3)	Column	0.75	COMB1	0.841477	COMB1	0.00042	COMB1	0.00042
676	column 30x30 8-dia12(3)	Column	0.75	COMB1	0.841477	COMB1	0.00042	COMB1	0.00042
676	column 30x30 8-dia12(3)	Column	1.5	COMB1	0.852469	COMB1	0.00042	COMB1	0.00042
676	column 30x30 8-dia12(3)	Column	3	COMB1	0.818445	COMB1	0.00042	COMB1	0.00042
676	column 30x30 8-dia12(3)	Column	0	COMB2	0.791121	COMB2	0.00042	COMB2	0.00042

676	column 30x30 8-dia12(3)	Column	0.75	COMB2	0.768346	COMB2	0.00042	COMB2	0.00042
676	column 30x30 8-dia12(3)	Column	0.75	COMB2	0.768346	COMB2	0.00042	COMB2	0.00042
676	column 30x30 8-dia12(3)	Column	1.5	COMB2	0.662406	COMB2	0.00042	COMB2	0.00042
676	column 30x30 8-dia12(3)	Column	3	COMB2	0.602171	COMB2	0.00042	COMB2	0.00042
673	column 30x30 16-dia20(6)	Column	0	COMB1	0.767238	COMB1	0.00042	COMB1	0.00042
673	column 30x30 16-dia20(6)	Column	0.75	COMB1	0.782355	COMB1	0.00042	COMB1	0.00042
673	column 30x30 16-dia20(6)	Column	0.75	COMB1	0.782355	COMB1	0.00042	COMB1	0.00042
673	column 30x30 16-dia20(6)	Column	1.5	COMB1	0.794151	COMB1	0.00042	COMB1	0.00042
673	column 30x30 16-dia20(6)	Column	3	COMB1	0.763057	COMB1	0.00042	COMB1	0.00042
673	column 30x30 16-dia20(6)	Column	0	COMB2	0.650569	COMB2	0.00042	COMB2	0.00042
673	column 30x30 16-dia20(6)	Column	0.75	COMB2	0.64058	COMB2	0.00042	COMB2	0.00042
673	column 30x30 16-dia20(6)	Column	0.75	COMB2	0.64058	COMB2	0.00042	COMB2	0.00042
673	column 30x30 16-dia20(6)	Column	1.5	COMB2	0.583731	COMB2	0.00042	COMB2	0.00042
673	column 30x30 16-dia20(6)	Column	3	COMB2	0.554157	COMB2	0.00042	COMB2	0.00042
673	column 35x35 12-dia20(6)	Column	0	COMB1	0.770451	COMB1	0.00049	COMB1	0.00049
673	column 35x35 12-dia20(6)	Column	0.75	COMB1	0.778202	COMB1	0.00049	COMB1	0.00049
673	column 35x35 12-dia20(6)	Column	0.75	COMB1	0.778202	COMB1	0.00049	COMB1	0.00049
673	column 35x35 12-dia20(6)	Column	1.5	COMB1	0.782144	COMB1	0.00049	COMB1	0.00049
673	column 35x35 12-dia20(6)	Column	3	COMB1	0.764855	COMB1	0.00049	COMB1	0.00049
673	column 35x35 12-dia20(6)	Column	0	COMB2	0.602714	COMB2	0.00049	COMB2	0.00049
673	column 35x35 12-dia20(6)	Column	0.75	COMB2	0.619211	COMB2	0.00049	COMB2	0.00049
673	column 35x35 12-dia20(6)	Column	0.75	COMB2	0.619211	COMB2	0.00049	COMB2	0.00049
673	column 35x35 12-dia20(6)	Column	1.5	COMB2	0.574596	COMB2	0.00049	COMB2	0.00049
673	column 35x35 12-dia20(6)	Column	3	COMB2	0.546216	COMB2	0.00049	COMB2	0.00049
673	column 40x40 8-dia14(6)	Column	0	COMB1	0.870937	COMB1	0.00056	COMB1	0.00056
673	column 40x40 8-dia14(6)	Column	0.75	COMB1	0.87429	COMB1	0.00056	COMB1	0.00056
673	column 40x40 8-dia14(6)	Column	0.75	COMB1	0.87429	COMB1	0.00056	COMB1	0.00056

673	column 40x40 8-dia14(6)	Column	1.5	COMB1	0.876153	COMB1	0.00056	COMB1	0.00056
673	column 40x40 8-dia14(6)	Column	3	COMB1	0.862756	COMB1	0.00056	COMB1	0.00056
673	column 40x40 8-dia14(6)	Column	0	COMB2	0.648663	COMB2	0.00056	COMB2	0.00056
673	column 40x40 8-dia14(6)	Column	0.75	COMB2	0.685543	COMB2	0.00056	COMB2	0.00056
673	column 40x40 8-dia14(6)	Column	0.75	COMB2	0.685543	COMB2	0.00056	COMB2	0.00056
673	column 40x40 8-dia14(6)	Column	1.5	COMB2	0.6413	COMB2	0.00056	COMB2	0.00056
673	column 40x40 8-dia14(6)	Column	3	COMB2	0.611767	COMB2	0.00056	COMB2	0.00056
670	column 40x40 16-dia20(9)	Column	0	COMB1	0.838872	COMB1	0.00056	COMB1	0.00056
670	column 40x40 16-dia20(9)	Column	0.75	COMB1	0.84253	COMB1	0.00056	COMB1	0.00056
670	column 40x40 16-dia20(9)	Column	0.75	COMB1	0.84253	COMB1	0.00056	COMB1	0.00056
670	column 40x40 16-dia20(9)	Column	1.5	COMB1	0.845033	COMB1	0.00056	COMB1	0.00056
670	column 40x40 16-dia20(9)	Column	3	COMB1	0.833327	COMB1	0.00056	COMB1	0.00056
670	column 40x40 16-dia20(9)	Column	0	COMB2	0.62167	COMB2	0.00056	COMB2	0.00056
670	column 40x40 16-dia20(9)	Column	0.75	COMB2	0.627322	COMB2	0.00056	COMB2	0.00056
670	column 40x40 16-dia20(9)	Column	0.75	COMB2	0.627322	COMB2	0.00056	COMB2	0.00056
670	column 40x40 16-dia20(9)	Column	1.5	COMB2	0.602821	COMB2	0.00056	COMB2	0.00056
670	column 40x40 16-dia20(9)	Column	3	COMB2	0.590024	COMB2	0.00056	COMB2	0.00056
670	column 45x45 12-dia20(9)	Column	0	COMB1	0.799728	COMB1	0.00063	COMB1	0.00063
670	column 45x45 12-dia20(9)	Column	0.75	COMB1	0.802373	COMB1	0.00063	COMB1	0.00063
670	column 45x45 12-dia20(9)	Column	0.75	COMB1	0.802373	COMB1	0.00063	COMB1	0.00063
670	column 45x45 12-dia20(9)	Column	1.5	COMB1	0.802527	COMB1	0.00063	COMB1	0.00063
670	column 45x45 12-dia20(9)	Column	3	COMB1	0.793104	COMB1	0.00063	COMB1	0.00063
670	column 45x45 12-dia20(9)	Column	0	COMB2	0.579717	COMB2	0.00063	COMB2	0.00063
670	column 45x45 12-dia20(9)	Column	0.75	COMB2	0.594151	COMB2	0.00063	COMB2	0.00063
670	column 45x45 12-dia20(9)	Column	0.75	COMB2	0.594151	COMB2	0.00063	COMB2	0.00063
670	column 45x45 12-dia20(9)	Column	1.5	COMB2	0.572787	COMB2	0.00063	COMB2	0.00063
670	column 45x45 12-dia20(9)	Column	3	COMB2	0.561048	COMB2	0.00063	COMB2	0.00063

TABLE: Concrete Details for Sap2000 output for material model 3 - Column Summary Data - BS8110 97

Frame	DesignSect	DesignType	Location	PMMCombo	PMMRatio	VMajCombo	VMajRebar	VMinCombo	VMinRebar
Text	Text	Text	m	Text	Unitless	Text	m2/m	Text	m2/m
676	column 20x20 4-dia14(3)	Column	0	COMB1	0.822906	COMB1	0.0002	COMB1	0.0002
676	column 20x20 4-dia14(3)	Column	0.75	COMB1	0.977632	COMB1	0.0002	COMB1	0.0002
676	column 20x20 4-dia14(3)	Column	0.75	COMB1	0.977632	COMB1	0.0002	COMB1	0.0002
676	column 20x20 4-dia14(3)	Column	1.5	COMB1	1.101242	COMB1	0.0002	COMB1	0.0002
676	column 20x20 4-dia14(3)	Column	3	COMB1	0.819277	COMB1	0.0002	COMB1	0.0002
676	column 20x20 4-dia14(3)	Column	0	COMB2	1.223471	COMB2	0.0002	COMB2	0.0002
676	column 20x20 4-dia14(3)	Column	0.75	COMB2	1.003501	COMB2	0.0002	COMB2	0.0002
676	column 20x20 4-dia14(3)	Column	0.75	COMB2	1.003501	COMB2	0.0002	COMB2	0.0002
676	column 20x20 4-dia14(3)	Column	1.5	COMB2	0.834103	COMB2	0.0002	COMB2	0.0002
676	column 20x20 4-dia14(3)	Column	3	COMB2	0.698326	COMB2	0.0002	COMB2	0.0002
673	column 25x25 8-dia16(6)	Column	0	COMB1	0.829976	COMB1	0.00025	COMB1	0.00025
673	column 25x25 8-dia16(6)	Column	0.75	COMB1	0.883438	COMB1	0.00025	COMB1	0.00025
673	column 25x25 8-dia16(6)	Column	0.75	COMB1	0.883438	COMB1	0.00025	COMB1	0.00025
673	column 25x25 8-dia16(6)	Column	1.5	COMB1	0.942624	COMB1	0.00025	COMB1	0.00025
673	column 25x25 8-dia16(6)	Column	3	COMB1	0.826742	COMB1	0.00025	COMB1	0.00025
673	column 25x25 8-dia16(6)	Column	0	COMB2	0.801626	COMB2	0.00025	COMB2	0.00025
673	column 25x25 8-dia16(6)	Column	0.75	COMB2	0.751374	COMB2	0.00025	COMB2	0.00025
673	column 25x25 8-dia16(6)	Column	0.75	COMB2	0.751374	COMB2	0.00025	COMB2	0.00025
673	column 25x25 8-dia16(6)	Column	1.5	COMB2	0.684584	COMB2	0.00025	COMB2	0.00025
673	column 25x25 8-dia16(6)	Column	3	COMB2	0.626314	COMB2	0.00025	COMB2	0.00025
670	column 30x30 12-dia16(9)	Column	0	COMB1	0.81657	COMB1	0.0003	COMB1	0.0003
670	column 30x30 12-dia16(9)	Column	0.75	COMB1	0.840267	COMB1	0.0003	COMB1	0.0003
670	column 30x30 12-dia16(9)	Column	0.75	COMB1	0.840267	COMB1	0.0003	COMB1	0.0003
670	column 30x30 12-dia16(9)	Column	1.5	COMB1	0.85299	COMB1	0.0003	COMB1	0.0003

670	column 30x30 12-dia16(9)	Column	3	COMB1	0.813382	COMB1	0.0003	COMB1	0.0003
670	column 30x30 12-dia16(9)	Column	0	COMB2	0.677389	COMB2	0.0003	COMB2	0.0003
670	column 30x30 12-dia16(9)	Column	0.75	COMB2	0.653298	COMB2	0.0003	COMB2	0.0003
670	column 30x30 12-dia16(9)	Column	0.75	COMB2	0.653298	COMB2	0.0003	COMB2	0.0003
670	column 30x30 12-dia16(9)	Column	1.5	COMB2	0.609447	COMB2	0.0003	COMB2	0.0003
670	column 30x30 12-dia16(9)	Column	3	COMB2	0.588132	COMB2	0.0003	COMB2	0.0003

TABLE: Concrete Details for Sap2000 output for material model 4 - Column Summary Data - BS8110 97

Frame	**DesignSect**	**DesignType**	**Location**	**PMMCombo**	**PMMRatio**	**VMajCombo**	**VMajRebar**	**VMinCombo**	**VMinRebar**
Text	Text	Text	m	Text	Unitless	Text	m2/m	Text	m2/m
676	column 20x20 4-dia16(3)	Column	0	COMB1	0.835358	COMB1	0.00028	COMB1	0.00028
676	column 20x20 4-dia16(3)	Column	0.75	COMB1	0.977374	COMB1	0.00028	COMB1	0.00028
676	column 20x20 4-dia16(3)	Column	0.75	COMB1	0.977374	COMB1	0.00028	COMB1	0.00028
676	column 20x20 4-dia16(3)	Column	1.5	COMB1	1.094486	COMB1	0.00028	COMB1	0.00028
676	column 20x20 4-dia16(3)	Column	3	COMB1	0.832177	COMB1	0.00028	COMB1	0.00028
676	column 20x20 4-dia16(3)	Column	0	COMB2	1.20271	COMB2	0.00028	COMB2	0.00028
676	column 20x20 4-dia16(3)	Column	0.75	COMB2	1.004557	COMB2	0.00028	COMB2	0.00028
676	column 20x20 4-dia16(3)	Column	0.75	COMB2	1.004557	COMB2	0.00028	COMB2	0.00028
676	column 20x20 4-dia16(3)	Column	1.5	COMB2	0.825404	COMB2	0.00028	COMB2	0.00028
676	column 20x20 4-dia16(3)	Column	3	COMB2	0.702913	COMB2	0.00028	COMB2	0.00028
673	column 25x25 12-dia16(6)	Column	0	COMB1	0.814003	COMB1	0.00035	COMB1	0.00035
673	column 25x25 12-dia16(6)	Column	0.75	COMB1	0.859536	COMB1	0.00035	COMB1	0.00035
673	column 25x25 12-dia16(6)	Column	0.75	COMB1	0.859536	COMB1	0.00035	COMB1	0.00035
673	column 25x25 12-dia16(6)	Column	1.5	COMB1	0.913514	COMB1	0.00035	COMB1	0.00035
673	column 25x25 12-dia16(6)	Column	3	COMB1	0.810832	COMB1	0.00035	COMB1	0.00035
673	column 25x25 12-dia16(6)	Column	0	COMB2	0.781145	COMB2	0.00035	COMB2	0.00035
673	column 25x25 12-dia16(6)	Column	0.75	COMB2	0.735219	COMB2	0.00035	COMB2	0.00035

673	column 25x25 12-dia16(6)	Column	0.75	COMB2	0.735219	COMB2	0.00035	COMB2	0.00035
673	column 25x25 12-dia16(6)	Column	1.5	COMB2	0.666113	COMB2	0.00035	COMB2	0.00035
673	column 25x25 12-dia16(6)	Column	3	COMB2	0.610702	COMB2	0.00035	COMB2	0.00035
670	column 30x30 12-dia20(9)	Column	0	COMB1	0.791562	COMB1	0.00042	COMB1	0.00042
670	column 30x30 12-dia20(9)	Column	0.75	COMB1	0.807998	COMB1	0.00042	COMB1	0.00042
670	column 30x30 12-dia20(9)	Column	0.75	COMB1	0.807998	COMB1	0.00042	COMB1	0.00042
670	column 30x30 12-dia20(9)	Column	1.5	COMB1	0.821559	COMB1	0.00042	COMB1	0.00042
670	column 30x30 12-dia20(9)	Column	3	COMB1	0.788472	COMB1	0.00042	COMB1	0.00042
670	column 30x30 12-dia20(9)	Column	0	COMB2	0.649969	COMB2	0.00042	COMB2	0.00042
670	column 30x30 12-dia20(9)	Column	0.75	COMB2	0.627601	COMB2	0.00042	COMB2	0.00042
670	column 30x30 12-dia20(9)	Column	0.75	COMB2	0.627601	COMB2	0.00042	COMB2	0.00042
670	column 30x30 12-dia20(9)	Column	1.5	COMB2	0.586697	COMB2	0.00042	COMB2	0.00042
670	column 30x30 12-dia20(9)	Column	3	COMB2	0.56537	COMB2	0.00042	COMB2	0.00042

11-Annex-2

Sap2000 V.14 Internal forces output for the different material models.

TABLE:-Sap output for Material model 1.

No of stories supported.	Column type.	Axial load in (KN.)	Location in (m.)	Moment in (KNm.)	Shear force in (KN.)	Deflection in (mm.)
9		-1656.82	0	19.52	60.94	0
			0.75	-25.74	60.94	-0.00019
			1.5		-14.06	-0.000234
			3	5.88	-14.06	0
9		-1673.48	0	16.23	60.33	0
			0.75	-29.01	60.33	-0.000145
			1.5		-14.67	-0.000175
			3	3.997	-14.67	0
9		-1686.77	0	12.74	59.77	0
			0.75	-32.1	59.77	-0.000111
			1.5		-15.23	-0.000133
			3	2.16	-15.23	0
6		-1148.86	0	23.16	61.03	0
			0.75	-22.52	61.03	-0.000471
			1.5		-13.975	-0.000541
			3	8.83	-13.975	0
6		-1165.11	0	19.15	60.31	0
			0.75	-26.08	60.31	-0.000327
			1.5		-14.69	-0.000395
			3	6.97	-14.69	0
6		-1185.55	0	15.21	59.76	0
			0.75	-29.61	59.76	-0.000236
			1.5		-15.24	-0.000288
			3	4.68	-15.24	0
3		-661.61	0	27.15	62.27	0
			0.75	-19.56	62.27	-0.00076
			1.5		-12.73	-0.000914
			3	9.07	-12.73	0
3		-671.49	0	23.78	61.69	0
			0.75	-22.5	61.69	-0.00048
			1.5		-13.31	-0.000584
			3	7.47	-13.31	0

TABLE:-Sap output for Material model 2.

No of stories supported.	Column type.	Axial load in (KN.)	Location in (m.)	Moment in (KNm.)	Shear force in (KN.)	Deflection in (mm.)
9		-1681.51	0	16.23	60.3	0
			0.75	-29.01	60.3	-0.000145
			1.5		-14.67	-0.00017
			3	4	-14.67	0
9			0	19.95	60.94	0
			0.75	-25.75	60.94	-0.000191
			1.5		-14.06	-0.000227
			3	5.88	-14.06	0
6			0	15.24	59.77	0
			0.75	-29.59	59.77	-0.000236
			1.5		-15.23	-0.000282
			3	4.67	-15.23	0
6		-1165.11	0	19.15	60.32	0
			0.75	-26.08	60.32	-0.000327
			1.5		-14.68	-0.000385
			3	6.97	-14.68	0
6		-1185.55	0	23.17	61.03	0
			0.75	-22.6	61.03	-0.000471
			1.5		-13.97	-0.000548
			3	8.83	-13.97	0
3		-661.61	0	23.82	61.71	0
			0.75	-22.46	61.71	-0.00048
			1.5		-13.29	-0.000576
			3	7.45	-13.29	0
3		-671.49	0	27.17	61.69	0
			0.75	-19.54	61.69	-0.00076
			1.5		-13.31	-0.000896
			3	9.06	-13.31	0

TABLE:-Sap output for Material model 3.

No of stories supported.	Column type.	Axial load in (KN.)	Location in (m.)	Moment in (KNm.)	Shear force in (KN.)	Deflection in (mm.)
9		-1584	0	25.69	61.97	0
			0.75	-20.79	61.97	-0.000333
			1.5		-13.03	-0.000389
			3	8.53	-13.03	0

6		-1102	0	25.83	61.62	0
			0.75	-20..383	61.62	-0.000236
			1.5		-13.38	-0.000282
			3	9.72	-13.38	0
3		-622	0	29.14	62.69	0
			0.75	-17.87	62.69	-0.000124
			1.5		-12.31	-0.000145
			3	9.83	-12.31	0

TABLE:-Sap output for Material model 4.

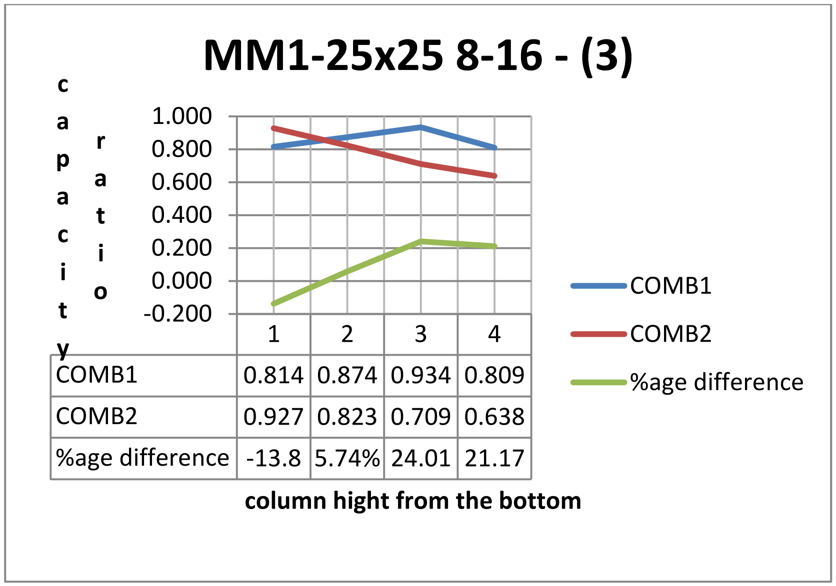
No of stories supported.	Column type.	Axial load in (KN.)	Location in (m.)	Moment in (KNm.)	Shear force in (KN.)	Deflection in (mm.)
9		-1584.34	0	25.69	61.97	0
			0.75	-20.79	61.97	-0.000333
			1.5		-13.03	-0.000382
			3	8.53	-13.03	0
6		-1110.25	0	25.84	61.62	0
			0.75	-20.38	61.62	-0.000649
			1.5		-13.37	-0.00075
			3	9.72	-13.37	0
3		-661.61	0	29.14	62.69	0
			0.75	-17.87	62.69	-0.000235
			1.5		-13.31	-0.000146
			3	9.38	-13.31	0

12-Annex-3

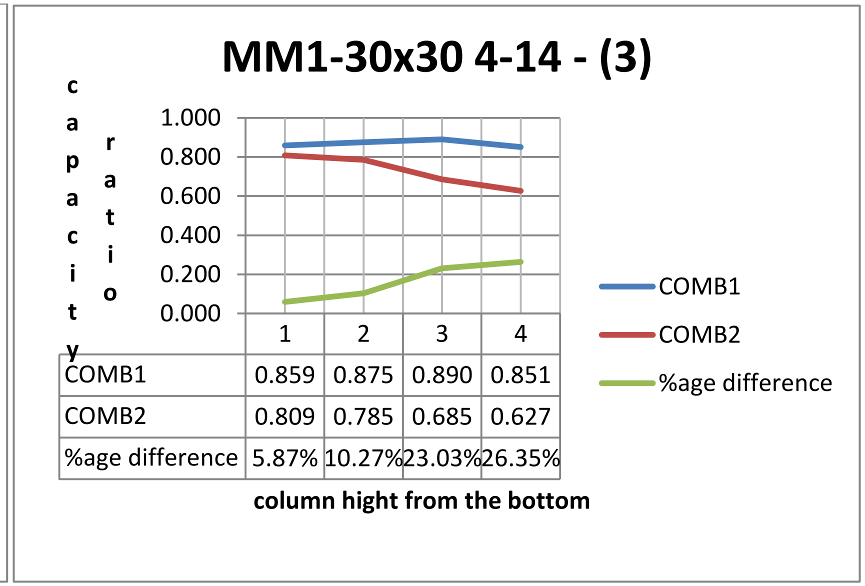
Graphical comparison of Ultimate (Gravity) load case and Accidental load case.

Graph for Material Model 1

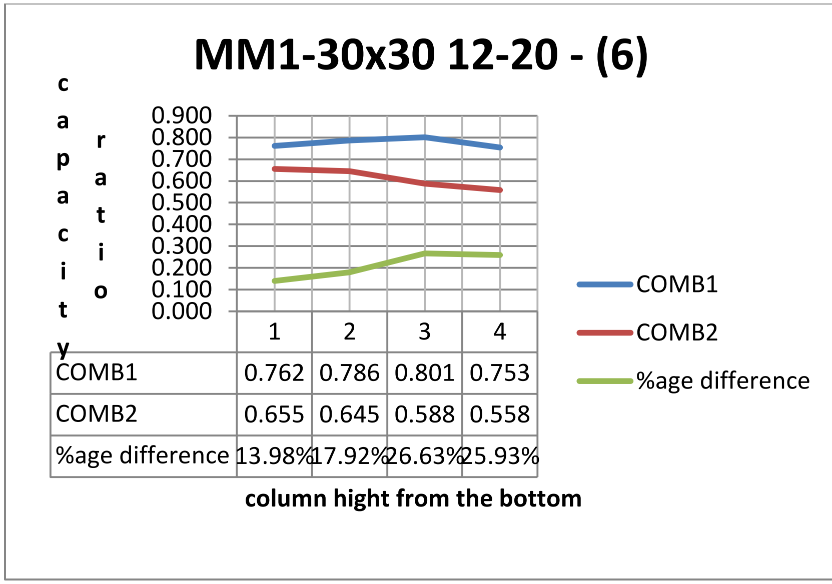
N.B:- In horizontl data of column height measured from the ground 1=0.75m, 2=1.5m, 3=2.25m & 4=3m



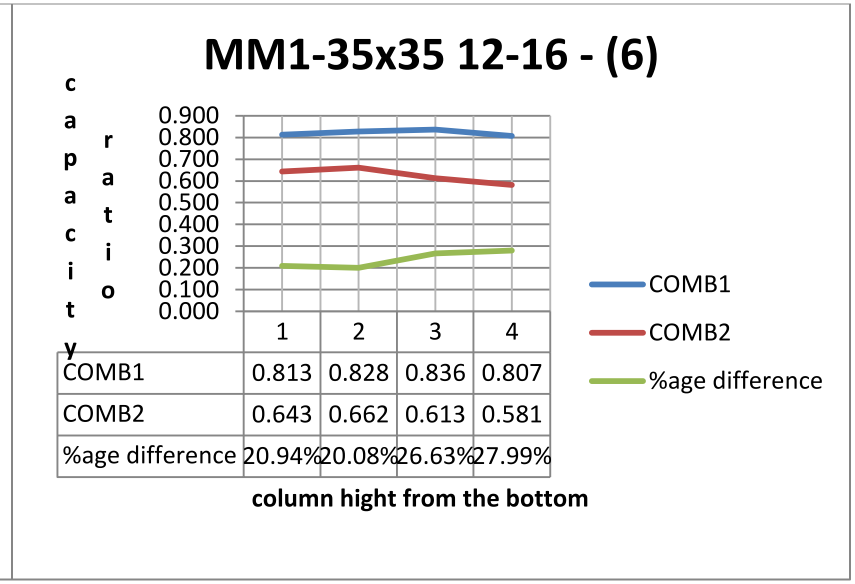
	1	2	3	4
COMB1	0.814	0.874	0.934	0.809
COMB2	0.927	0.823	0.709	0.638
%age difference	-13.8	5.74%	24.01	21.17



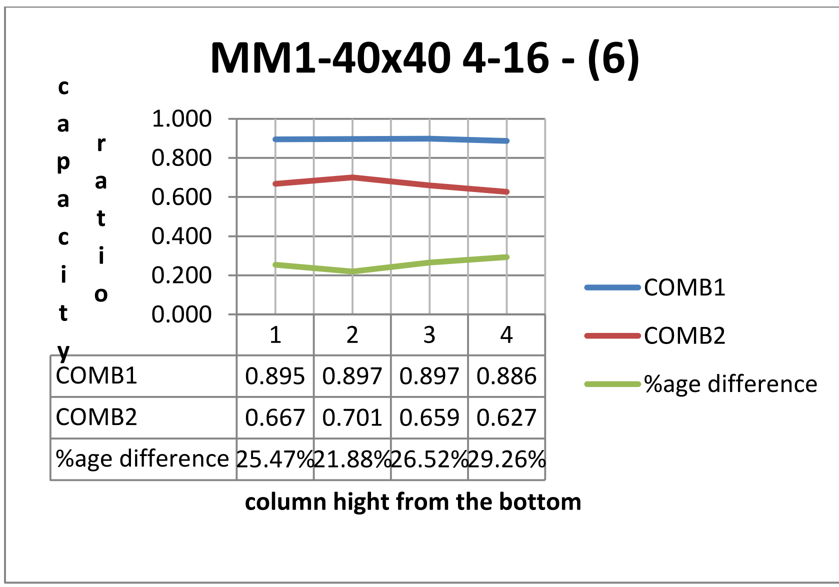
	1	2	3	4
COMB1	0.859	0.875	0.890	0.851
COMB2	0.809	0.785	0.685	0.627
%age difference	5.87%	10.27%	23.03%	26.35%



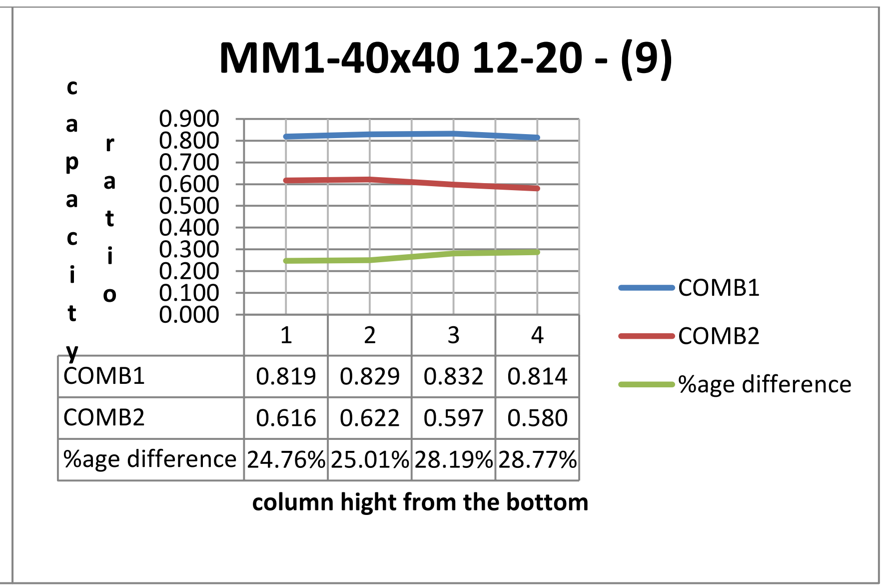
	1	2	3	4
COMB1	0.762	0.786	0.801	0.753
COMB2	0.655	0.645	0.588	0.558
%age difference	13.98%	17.92%	26.63%	25.93%



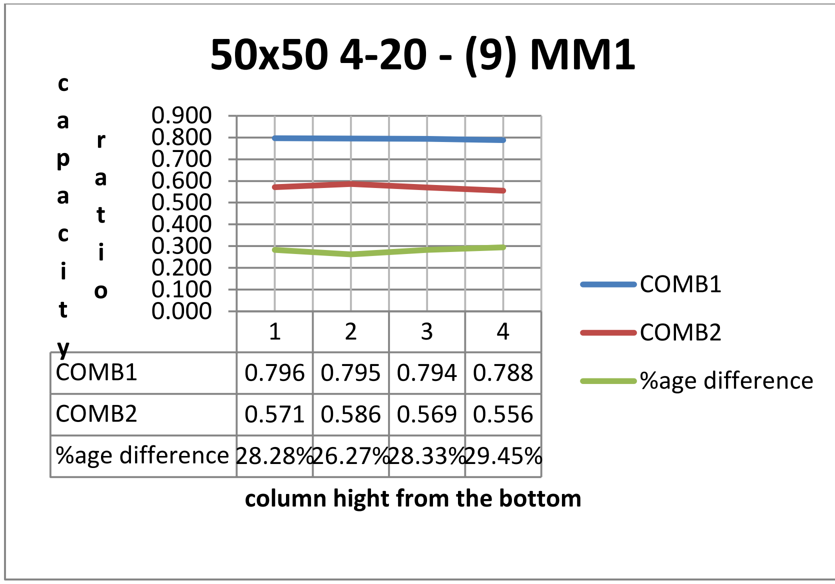
	1	2	3	4
COMB1	0.813	0.828	0.836	0.807
COMB2	0.643	0.662	0.613	0.581
%age difference	20.94%	20.08%	26.63%	27.99%



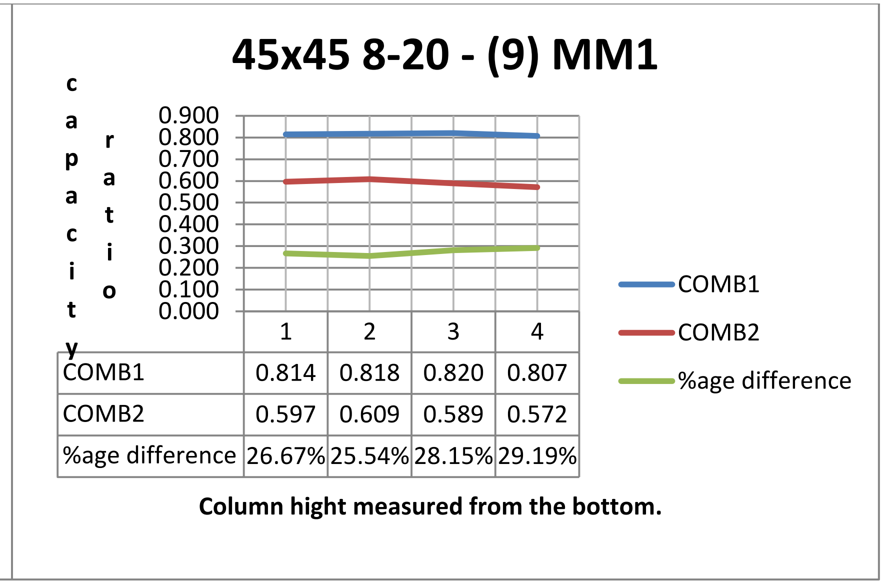
	1	2	3	4
COMB1	0.895	0.897	0.897	0.886
COMB2	0.667	0.701	0.659	0.627
%age difference	25.47%	21.88%	26.52%	29.26%



	1	2	3	4
COMB1	0.819	0.829	0.832	0.814
COMB2	0.616	0.622	0.597	0.580
%age difference	24.76%	25.01%	28.19%	28.77%

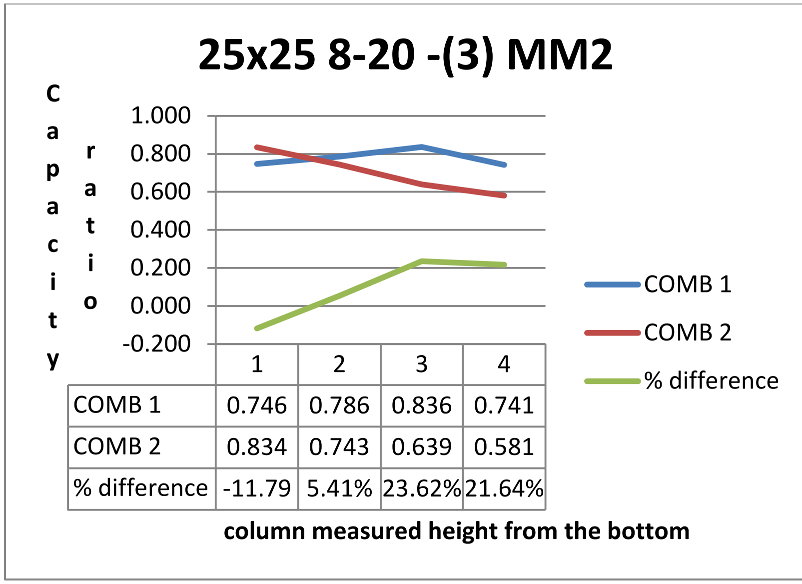


	1	2	3	4
COMB1	0.796	0.795	0.794	0.788
COMB2	0.571	0.586	0.569	0.556
%age difference	28.28%	26.27%	28.33%	29.45%

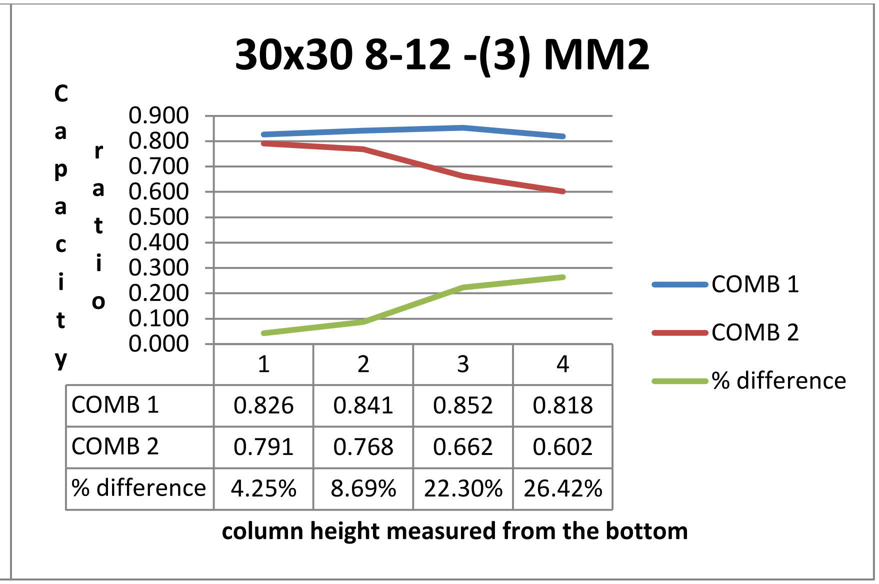


	1	2	3	4
COMB1	0.814	0.818	0.820	0.807
COMB2	0.597	0.609	0.589	0.572
%age difference	26.67%	25.54%	28.15%	29.19%

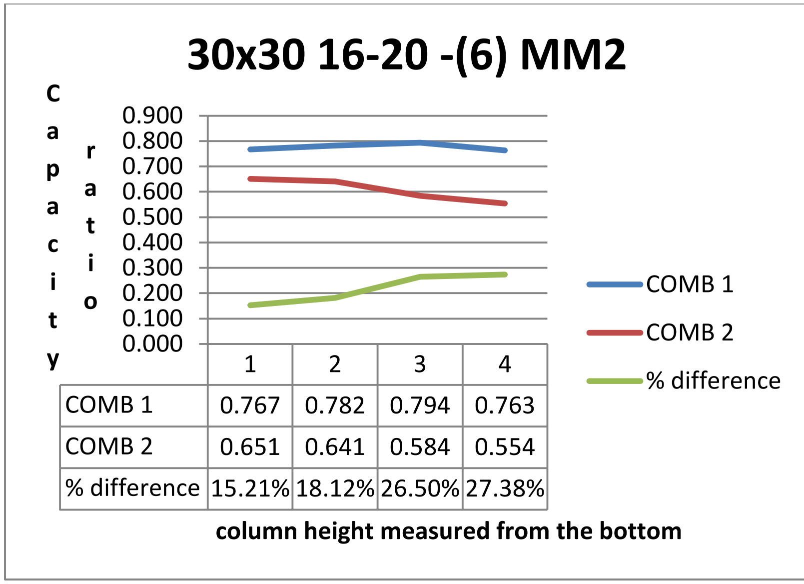
Graph for material model 2



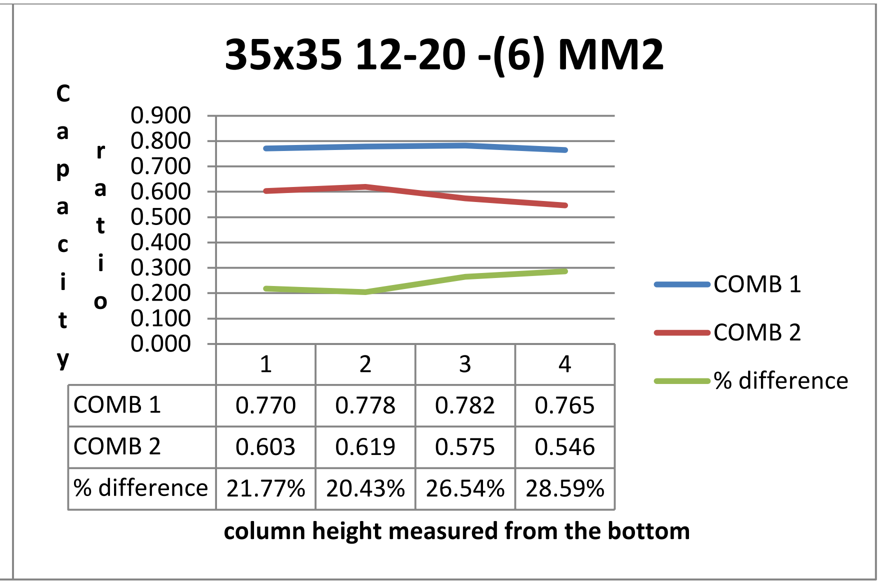
	1	2	3	4
COMB 1	0.746	0.786	0.836	0.741
COMB 2	0.834	0.743	0.639	0.581
% difference	-11.79	5.41%	23.62%	21.64%



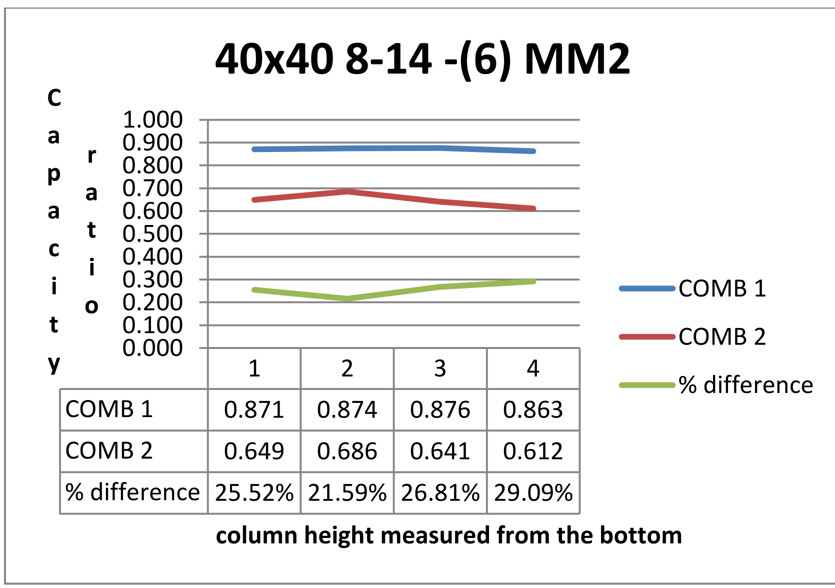
	1	2	3	4
COMB 1	0.826	0.841	0.852	0.818
COMB 2	0.791	0.768	0.662	0.602
% difference	4.25%	8.69%	22.30%	26.42%



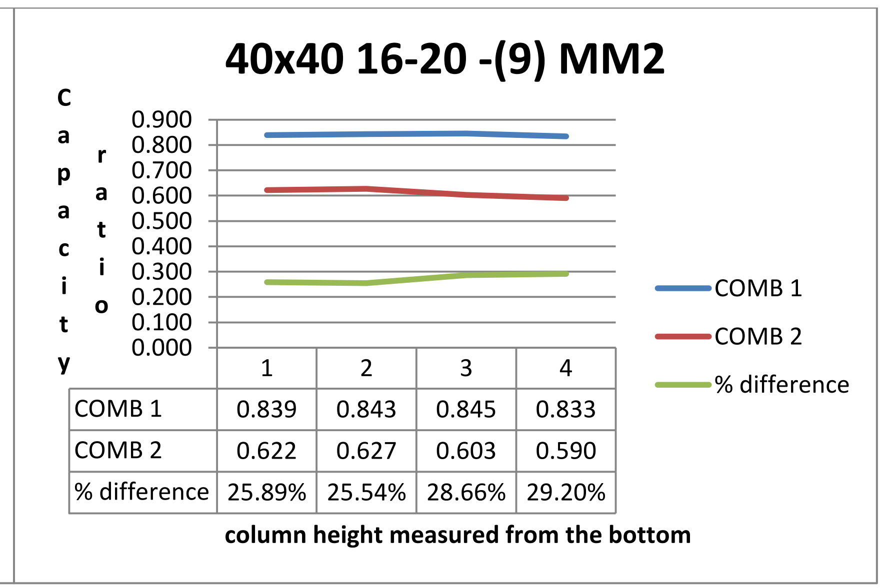
	1	2	3	4
COMB 1	0.767	0.782	0.794	0.763
COMB 2	0.651	0.641	0.584	0.554
% difference	15.21%	18.12%	26.50%	27.38%



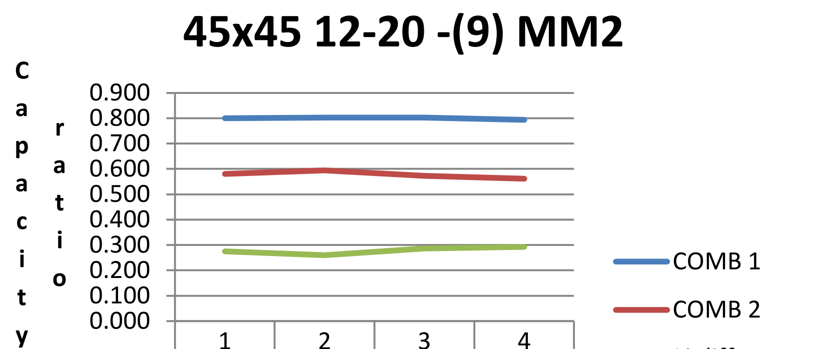
	1	2	3	4
COMB 1	0.770	0.778	0.782	0.765
COMB 2	0.603	0.619	0.575	0.546
% difference	21.77%	20.43%	26.54%	28.59%



	1	2	3	4
COMB 1	0.871	0.874	0.876	0.863
COMB 2	0.649	0.686	0.641	0.612
% difference	25.52%	21.59%	26.81%	29.09%

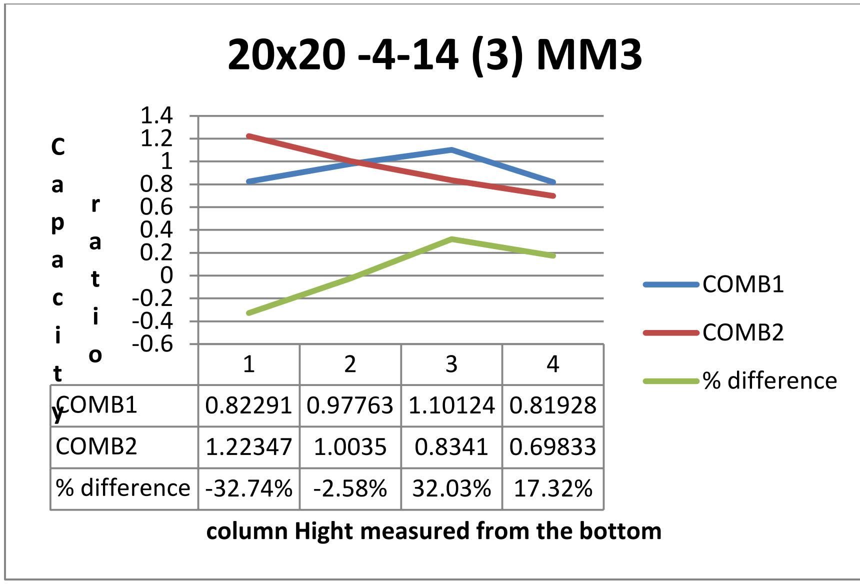


	1	2	3	4
COMB 1	0.839	0.843	0.845	0.833
COMB 2	0.622	0.627	0.603	0.590
% difference	25.89%	25.54%	28.66%	29.20%

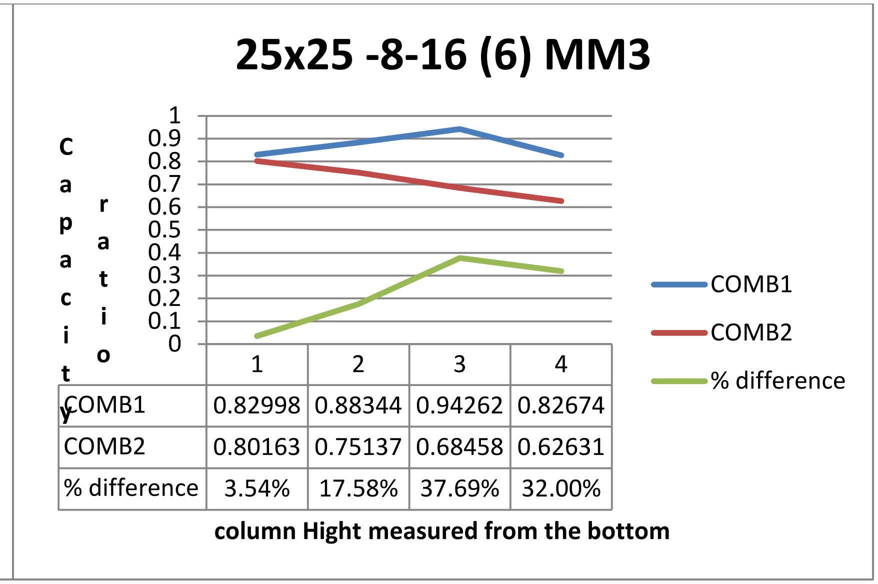


	1	2	3	4
COMB 1	0.800	0.802	0.803	0.793
COMB 2	0.580	0.594	0.573	0.561
% difference	27.51%	25.95%	28.63%	29.26%

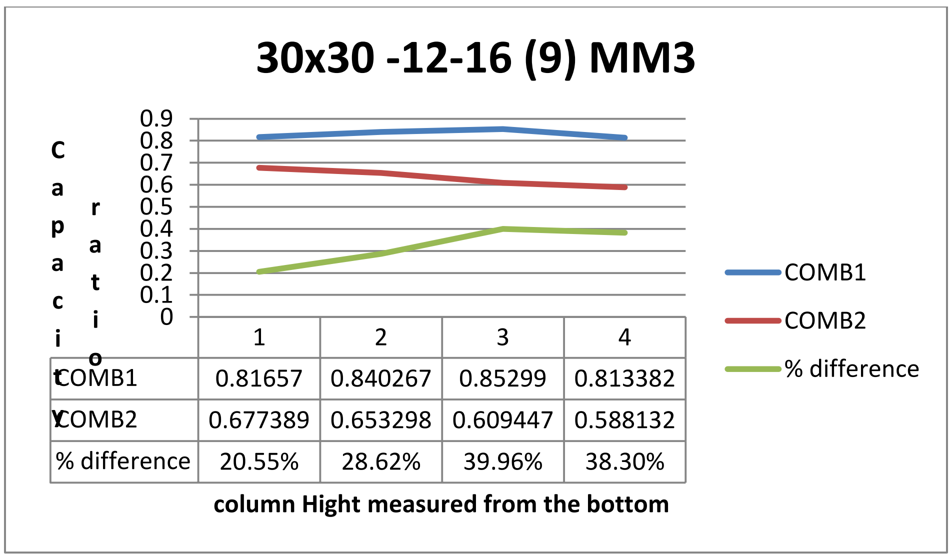
Graph for material model 3



	1	2	3	4
COMB1	0.82291	0.97763	1.10124	0.81928
COMB2	1.22347	1.0035	0.8341	0.69833
% difference	-32.74%	-2.58%	32.03%	17.32%

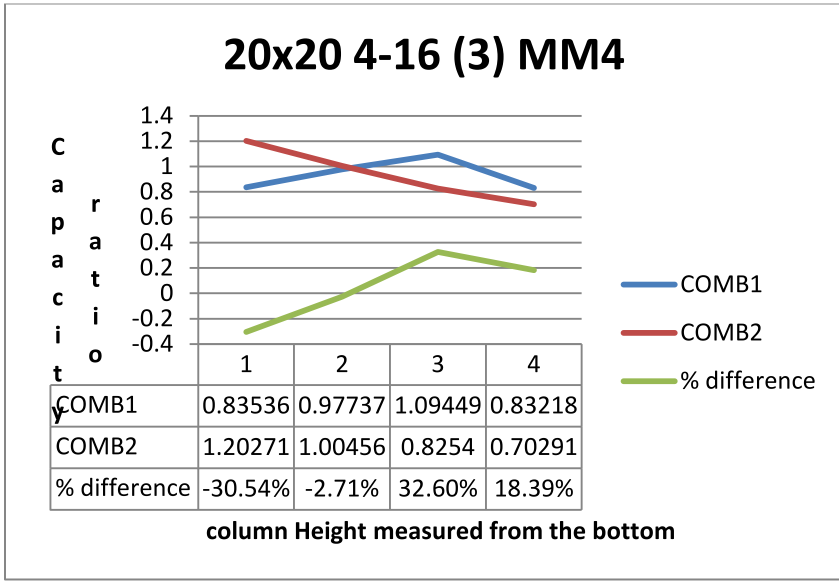


	1	2	3	4
COMB1	0.82998	0.88344	0.94262	0.82674
COMB2	0.80163	0.75137	0.68458	0.62631
% difference	3.54%	17.58%	37.69%	32.00%

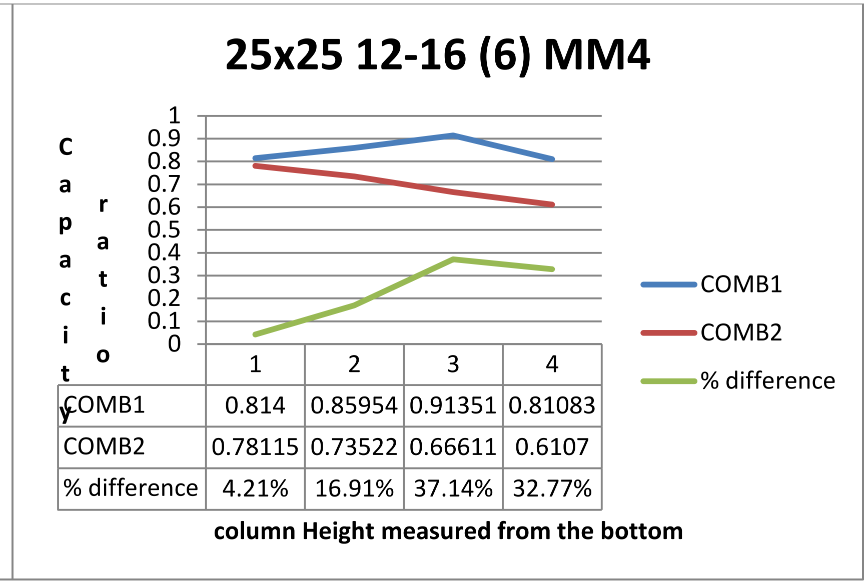


	1	2	3	4
COMB1	0.81657	0.840267	0.85299	0.813382
COMB2	0.677389	0.653298	0.609447	0.588132
% difference	20.55%	28.62%	39.96%	38.30%

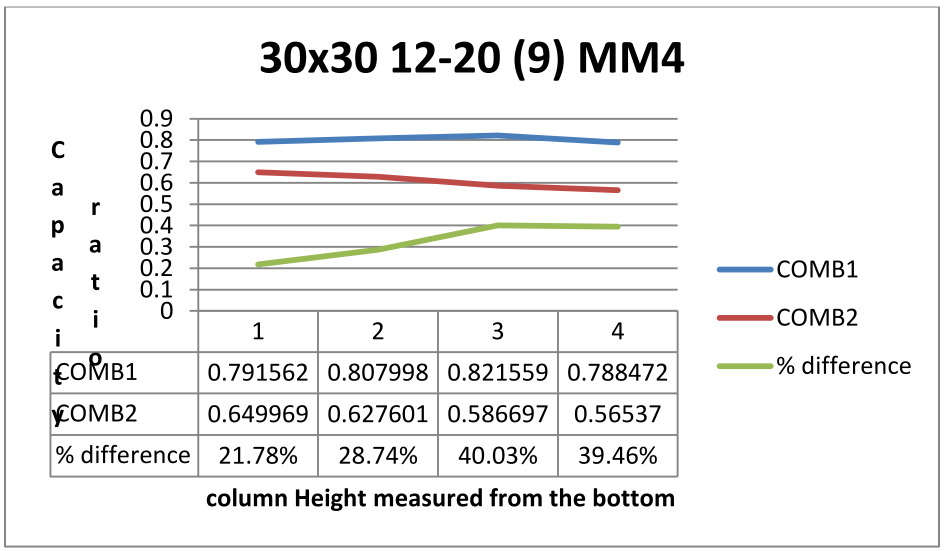
Graph for material model 4



	1	2	3	4
COMB1	0.83536	0.97737	1.09449	0.83218
COMB2	1.20271	1.00456	0.8254	0.70291
% difference	-30.54%	-2.71%	32.60%	18.39%



	1	2	3	4
COMB1	0.814	0.85954	0.91351	0.81083
COMB2	0.78115	0.73522	0.66611	0.6107
% difference	4.21%	16.91%	37.14%	32.77%



	1	2	3	4
COMB1	0.791562	0.807998	0.821559	0.788472
COMB2	0.649969	0.627601	0.586697	0.56537
% difference	21.78%	28.74%	40.03%	39.46%