EVALUATION OF RESPONSE REDUCTION FACTORS FOR MOMENT RESISTING RC FRAMES

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CERTIFICATE

This is to certify that the thesis entitled "**Evaluation of Response Reduction Factors for Moment Resisting RC Frames"** submitted by **Minnu M M** in partial fulfilment of the requirement for the award of **Master of Technology** degree in **Civil Engineering** with specialization in **Structural Engineering** to the National Institute of Technology, Rourkela is an authentic record of research work carried out by her under my supervision. The contents of this thesis, in full or in parts, have not been submitted to any other Institute or University for the award of any degree or diploma.

Project Guide

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ABSTRACT

Keywords: *OMRF, SMRF, Response Reduction Factor, Pushover, Ductility, Confinement models*

Moment resisting frames are commonly used as the dominant mode of lateral resisting system in seismic regions for a long time. The poor performance of Ordinary Moment Resisting Frame (OMRF) in past earthquakes suggested special design and detailing to warrant a ductile behaviour in seismic zones of high earthquake (zone III, IV & V). Thus when a large earthquake occurs, Special Moment Resisting Frame (SMRF) which is specially detailed with a response reduction factor, $R = 5$ is expected to have superior ductility. The response reduction factor of 5 in SMRF reduces the design base shear and in such a case these building rely greatly on their ductile performance. To ensure ductile performance, this type of frames shall be detailed in a special manner recommended by IS 13920. The objective of the present study is to evaluate the R factors of these frames from their nonlinear base shear versus roof displacement curves (pushover curves) and to check its adequacy compared to code recommended R value.

The accurate estimation of strength and displacement capacity of nonlinear pushover curves requires the confinement modelling of concrete as per an accepted confinement model. A review of various concrete confinement models is carried out to select appropriate concrete confinement model. It is found that modified Kent and Park model is an appropriate model and it is incorporated in the modelling of nonlinearity in concrete sections. The frames with number of storeys 2, 4, 8, and 12 (with four bays) are designed and detailed as SMRF and OMRF as per IS 1893 (2002). The pushover curves of each SMRF and OMRF frames are generated and converted to a bilinear format to calculate the behaviour factors. The response reduction factors obtained show in general that both the OMRF and SMRF frames, failed to achieve the respective target values of response reduction factors recommended by IS 1893 (2002) marginally. The components of response reduction factors such as over-strength and ductility factors also evaluated for all the SMRF and OMRF frames. It was also found that shorter frames exhibit higher R factors and as the height of the frames increases the R factors decreases.

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ABBREVIATIONS

OMRF	Ordinary Moment Resisting Frames
SMRF	Special Moment Resisting Frames
RC	Reinforced Concrete
IS	Indian Standard
FEMA	Federal Emergency Management Agency
ATC	Applied Technology Council

NOTATIONS

f_{ck}	Characteristic strength of concrete
fc'	Cylinder Strength of Concrete
E_c	Young's Modulus of Concrete
f_y	Yield Strength of Transverse Steel
f'_{co}	Unconfined Peak strength of concrete
ε_{co}	Strain corresponding to unconfined peak stress of concrete
f'_{cc}	Peak Stress of concrete
ε_{cc}	Strain Corresponding to peak stress
ε_{cu}	Critical compressive strain
ρ_s	Volumetric ratio of confining steel
f_{yh}	Grade of confining steel
ε_{sm}	Steel strain at maximum tensile stress
k_e	Confinement effectiveness coefficient
K	Confinement Factor
s	Spacing of Hoops
V_d	Design Base Shear
W	Seismic weight of the building
A_h	Design Horizontal Seismic Co-efficient
T_a	Time period for moment resisting frames without infill
H	Height of the building in metres
S_a/g	Spectral Acceleration Co-efficient

$\boldsymbol{Q_i}$	Design lateral force at floor *i*
$\boldsymbol{W_i}$	W_i = Seismic weight of floor i
$\boldsymbol{h_i}$	h_i = Height of floor i
$\boldsymbol{N}$	No. of storeys in the building
$\boldsymbol{R}$	Response Reduction Factor
$\boldsymbol{R_s}$	Strength factor
$\boldsymbol{R_\mu}$	Ductility factor
$\boldsymbol{R}_\xi$	Damping factor
$\boldsymbol{R_R}$	Redundancy factor
$\boldsymbol{V_u}$	Ultimate base shear of the inelastic response
$\boldsymbol{V_e}$	Base shear of the elastic response
$\boldsymbol{\mu}$	ductility capacity
$\boldsymbol{\Delta_u}$	Maximum displacement
$\boldsymbol{\Delta_y}$	Yield Displacement

CHAPTER 1

INTRODUCTION

1.1 CONCRETE CONFINEMENT

Column shear failure has been identified as the frequently mentioned cause of concrete structure failure and downfall during the past earthquakes. In the earthquake resistant design of reinforced concrete sections of buildings, the plastic hinge regions should be strictly detailed for ductility in order to make sure that severe ground shaking during earthquakes will not cause collapse of the structure. The most important design consideration for ductility in plastic hinge regions of reinforced concrete columns is the provision of adequate transverse reinforcement in the form of spirals or circular hoops or of rectangular arrangements of steel. The cover concrete will be unconfined and will eventually become ineffective after the compressive strength is attained, but the core concrete will continue to carry stress at high strains. Transverse reinforcements which are mainly provided for resisting shear force, helps in confining the core concrete and prevents buckling of the longitudinal bars. The core concrete which remains confined by the transverse reinforcement is not permitted to dilate in the transverse direction, thereby helps in the enhancement of its peak strength and ultimate strain capacities. Thus confinement of concrete by suitable arrangements of transverse reinforcement results in a significant increase in both the strength and the ductility of compressed concrete.

Confining reinforcements are mainly provided at the column and beam ends and beam-column joints. The hoops should enclose the whole cross section excluding the cover concrete and must be closed by 135° hooks embedded in the core concrete, this prevents opening of the hoops if spalling of the cover concrete occurs. Seismic codes recommend the use of closely spaced transverse reinforcement in-order to confine the concrete and prevent buckling of longitudinal reinforcement.

Ductile response demands that elements yield in flexure and shear failure has to be prevented. Shear failure in columns, is relatively brittle and can lead to immediate loss of lateral strength and stiffness. To attain a ductile nature, special design and detailing of the RC sections is required. IS 13920 recommends certain standards for the provision of confining reinforcements for beams and columns. The code suggests that the primary step is to identify the regions of yielding, design those sections for adequate moment capacity, and then estimate design shears founded on equilibrium supposing the flexural yielding sections improve credible moment strengths. The probable moment capacity is considered using methods that give a higher estimate of the moment strength of the planned cross section. Transverse reinforcement provision given in IS 13920 is given in Figures 1.1 a, 1.1 b and 1.2 for Columns and beams.

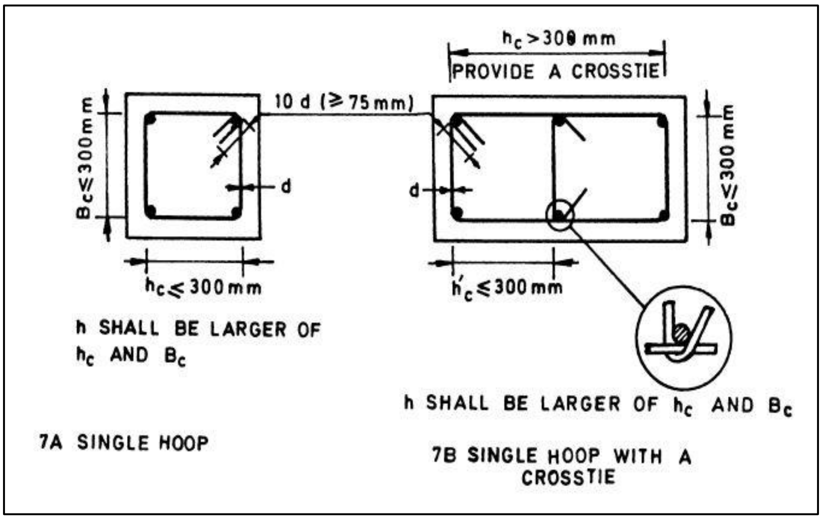


Fig 1.1 (a)

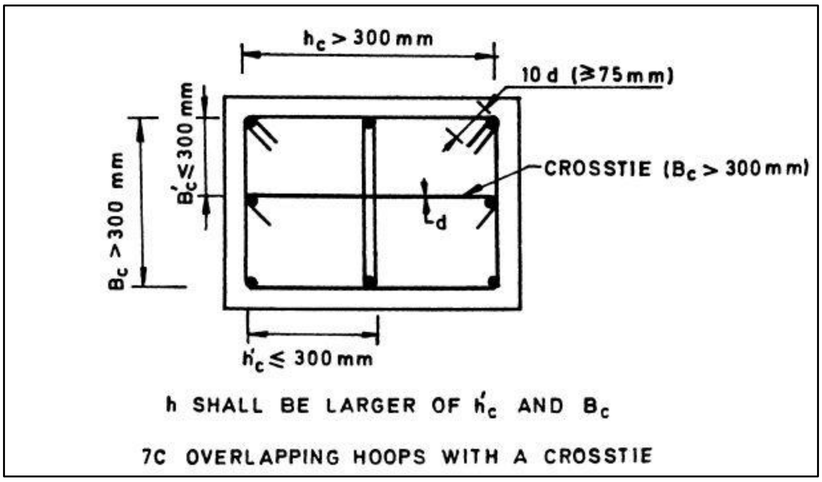


Fig 1.1 b

Transverse Reinforcement in columns (Reference: IS 13920(2002))

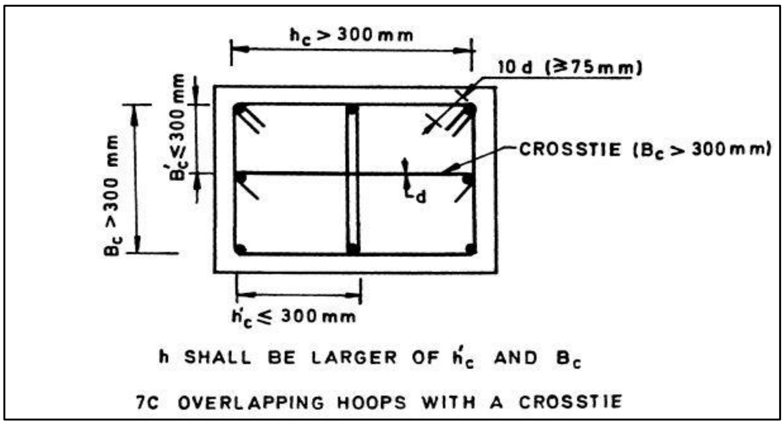


Fig 1.2 – Shear Reinforcement in beams (Reference: IS 13920(2002))

1.2 CONFINEMENT MODELS FOR CONCRETE

Various models for the stress-strain relation of concrete have been suggested in the past. Though the performance of concrete up to the peak concrete strength is well established, the post-peak part and the behaviour of high-strength concrete have not been explored.

A proper stress-strain relation for confined concrete is required. Confinement in concrete is attained by the suitable provision of transverse reinforcement. At small intensities of stress, transverse reinforcement is barely stressed; the concrete behaves much like unconfined concrete. At stresses near to the uniaxial strength of concrete interior fracturing leads the concrete to expand and bear out versus the transverse reinforcement which causes a confining action in the concrete. This occurrence of confining concrete by appropriate arrangement of transverse reinforcement grounds a significant hike in the strength and ductility of concrete. The improvement of strength and ductility by confining the concrete is a significant feature that needs to be reflected in the design of structural concrete elements particularly in areas susceptible to seismic activity. Again, several models are available for the stress-strain relation of confined concrete.

In this study different models are taken into account and studied. IS code provides a stress-strain relation which does not consider any effect of confinement. Other models that were developed which evaluated the stress strain relation considering the confinement effect were Kent and Park model (1971), Modified Kent and Park model (Scott 1982), Mander's model (Mander 1988a, 1988b), Razvi Model (Saatcioglu and Razvi 1992) etc. Detailed explanations of each model are given in Chapter 3. In this project Modified Kent and Park model is used, as this model shows the highest percentage increase in column capacity and ductility and is more close to Indian conditions.

1.3 SPECIAL AND ORDNARY MOMENT RESISTING FRAMES (SMRF AND OMRF)

According to Indian standards moment resisting frames are classified as Ordinary Moment Resisting Frames (OMRF) and Special Moment Resisting Frames (SMRF) with response reduction factors 3 and 5 respectively. Another main difference is the provision of ductile detailing according to IS 13920 as explained in Section 1.1 for the SMRF structures. The differences between these two are given in Table 1.1. Different international codes classify buildings in different ways which are elaborated in Section 2.2.

Table 1.1 Differences between SMRF and OMRF

SMRF	**OMRF**
It is a moment-resisting frame specially detailed to provide ductile behaviour and comply with the requirements given in IS 13920.	It is a moment-resisting not meeting special detailing requirement for ductile behavior.
Used under moderate-high earthquakes	Used in low earthquakes
R = 5	R = 3
Low design base shear.	High design base shear.
It is safe to design a structure with ductile detailing.	It is not safe to design a structure without ductile detailing.

1.4 RESPONSE REDUCTION FACTOR (*R*)

It is the factor by which the actual base shear forces, that would be generated if the structure were to remain elastic during its response to the Design Basis Earthquake (DBE) shaking, shall be reduced to obtain the design lateral force (IS 1893 Part 1, 2002).This factor permits a designer to use a linear elastic force-based design while accounting for non-linear behaviour and deformation limits. Response reduction factor of 3 is used for OMRF and 5 for SMRF during the building design. In this project four different RC plain frames designed as both OMRF and SMRF is considered and its response reduction factors are calculated by using non-linear static analysis. Detailed steps involved in calculation of *R* are given in Chapter 4.

1.5 MOTIVATION AND OBJECTIVES OF THE PRESENT STUDY

Moment-resisting frames are commonly used in urban areas worldwide as the dominant mode of building construction. However, documented poor performance of ordinary moment frames in past earthquakes warned the international community that this structural system required special design and detailing in order to warrant a ductile behaviour when subjected to the action of strong earthquake. When large earthquake occurs, SMRF is expected to have superior ductility and provide superior energy dissipation capacity. Current design provisions assigned the highest *R* factor to SMRF. The elastic forces are reduced by a response reduction factor to calculate the seismic design base shear. The building shall be detailed as Special Moment Resisting Frames (SMRF) if the value of *R* assumed is 5. Once the design is being done, it is required to ensure that the designed building exhibit the adequate behaviour factors or response reduction factors. Present study is an attempt to evaluate the response reduction factors of SMRF and OMRF frames and to check the adequacy of *R* factors used by IS code.

The broad objectives of the present study have been identified as follows:

- To review various existing Confinement Models for concrete
- To find response reduction factors (*R*) for frames designed as SMRF and OMRF according to IS 1893 (2002).
- To determine the over-strength and ductility factors for SMRF and OMRF frames

1.6 SCOPE OF WORK

The present study is limited RC plane frames without shear wall, basement, and plinth beam. The stiffness and strength of Infill walls is not considered. The soil structure interface effects are not taken into account in the study. The flexibility of floor diaphragms is ignored and is considered as stiff diaphragm. The column bases are assumed to be fixed in the study. OpenSees platform (McKenna *et al.*, 2000) is used in the present study. The non-linearity in the material properties are modeled using fiber models available in OpenSees platform.

1.7 ORGANISATION OF THE THESIS

Following this introductory chapter, the organisation of further Chapters is done as explained below.

i. A review of literature conducted on various fields like confinement models, ductility, pushover, and response reduction factor are provided in Chapter 2.
ii. Review of existing confinement models, details of various SMRF and OMRF frames and parametric study are discussed in Chapter 3.
iii. Modelling and nonlinear static pushover analysis of the SMRF and OMRF frames and calculation of response reduction factors are covered in Chapter 4.
iv. Finally in Chapter 5, discussion of results, limitations of the work and future scope of this study is dealt with.

Chapter 2
LITERATURE REVIEW

2.1 GENERAL

An extensive literature review was carried out prior to the project. The survey of literature includes classification of RC framed buildings, SMRF and OMRF, response reduction factor, various stress strain models and pushover analysis.

2.2 SMRF and OMRF

IS 1893 (Part 1), 2002.Criteria for earthquake resistant design of structures Part 1 General provisions and buildings, Bureau of Indian Standards (BIS) classifies RC frame buildings into two classes, Ordinary Moment Resisting Frames (OMRF) and Special Moment Resisting Frames (SMRF) with response reduction factors 3 and 5 respectively. Response Reduction Factor (R) is the factor by which the actual base shears that would be generated if the structure were to remain elastic during its response to the Design Basis Earthquake (DBE) shaking, shall be reduced to obtain the design lateral force.

ACI 318: Building code requirements for reinforced concrete and commentary, published by American Concrete Institute. ASCE 7 classifies RC frame buildings into three ductility classes: Ordinary Moment Resisting Frame (OMRF), Intermediate Moment Resisting Frames (IMRF) and Special Moment Resisting Frames (SMRF) and corresponding reduction factors are 3, 5 and 8, respectively.

Euro-code 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings, European Committee for Standardization, aims to ensure the protection of life during a major earthquake simultaneously with the restriction of damages during more frequent earthquakes. Euro-code 8 (EN 1998-1) classifies the building ductility as Ductility Class low (DCL) that does not require delayed ductility and the resistance to seismic loading is achieved through the capacity of the structure and reduction factor $q = 1.5$, Ductility Class Medium (DCM) that allows high levels of ductility and there are responsive design demands with reduction factor $1.5 < q < 4$ and Ductility Class High (DCH) that allows even higher levels of ductility and there are responsive strict and complicated design demands and reduction factor q>4.

Khose *et al.* (2012) performed an overview of ductile detailing requirements for RC frame buildings in different seismic design codes. The results obtained were as shown in Table 2.1.

Table 2.1: Ductile detailing Criteria as per different codes

○ Provision is not available

● Provision is available

Ductile Detailing Criteria		ASCE 7			Euro-code 8			IS 1893	
		OMRF	IMRF	SMRF	DCL	DCM	DCH	OMRF	SMRF
Capacity Design	Strong column Weak beam	○	○	●	○	●	●	○	○
	Capacity shear for column	○	●	●	○	●	●	○	●
	Capacity shear for beam	○	●	●	○	●	●	○	●
Special Confinement Reinforcement	Column	○	●	●	○	●	●	○	●
	Beam	○	●	●	○	●	●	○	●
Special Anchorage Reinforcement	Interior joint	○	○	●	○	●	●	○	●
	Exterior joint	○	○	●	○	●	●	○	●
Joint shear design		○	○	●	○	○	●	○	○

Han and Jee (2005) investigated the seismic behavior of columns in Ordinary Moment Resisting Frames (OMRF) and Intermediate Moment Resisting Frames (IMRF). In their study two three-story OMRF and IMRF were designed as per the minimum design and reinforcement detailing requirements suggested by ACI 318-02. The IMRF interior column specimens exhibited superior drift capacities compared to the OMRF column specimens. According to the test results, the OMRF and IMRF column specimens had drift capacities greater than 3.0% and 4.5%, respectively. Ductility capacity of OMRF and IMRF specimens exceeded 3.01 and 4.53, respectively.

Sadjadi *et al*. (2006), conducted an analytical study for assessing the seismic performance of RC frames using non-linear time history analysis and push-over analysis. A typical 5-story frame was designed as ductile, nominally ductile and GLD structures. Most of the RC frame structures built before 1970 and located in areas prone to seismic actions were designed only for gravity loads without taking into account the lateral loads. These structures were referred to as Gravity Load Designed (GLD) frames. The lack of seismic considerations in GLD structures resulted in non-ductile behavior in which the lateral load resistance of these buildings may be insufficient for even moderate earthquakes. It was concluded that both the ductile and the nominally ductile frames behaved very well under the considered earthquake, while the seismic performance of the GLD structure was not satisfactory. After the damaged GLD frame was retrofitted the seismic performance was improved.

Uma and Jain (2006) conducted a critical review of recommendations of well-established codes regarding design and detailing aspects of beam column joints. The codes of practice considered are ACI 318M-02, NZS 3101: Part 1:1995 and the Euro-code 8 of EN 1998-1:2003. It was observed that ACI 318M-02 requires smaller column depth as compared to the other two codes based on the anchorage conditions. NZS 3101:1995 and EN 1998-1:2003 consider the shear stress level to obtain the required stirrup reinforcement whereas ACI 318M-02 provides stirrup reinforcement to retain the axial load capacity of column by confinement. ACI requires transverse reinforcement in proportion to the strength of the concrete whereas NZS sets limits based on the level of nominal shear stress that is experienced by the joint core. EN provides shear reinforcement to confine the joint and to bring down the maximum tensile stress to design value. NZS and EN codes emphasize on provision of 135^0 hook on both ends of the cross-ties; whereas ACI code accepts 135^0 at one end and 90^0 hooks at the other end and insists on proper placement of stirrups to provide effective confinement. In general, the provisions of the NZS are most stringent, while the ACI code provisions are most liberal. The EN code has followed a somewhat middle path: in some respects it is more conservative like the NZS code, while in other respects it is closer to the ACI provisions. Therefore it was concluded that the EN code provisions are more likely to offer a good model to follow for the countries in the process of developing their own codes.

2.3 DUCTILITY

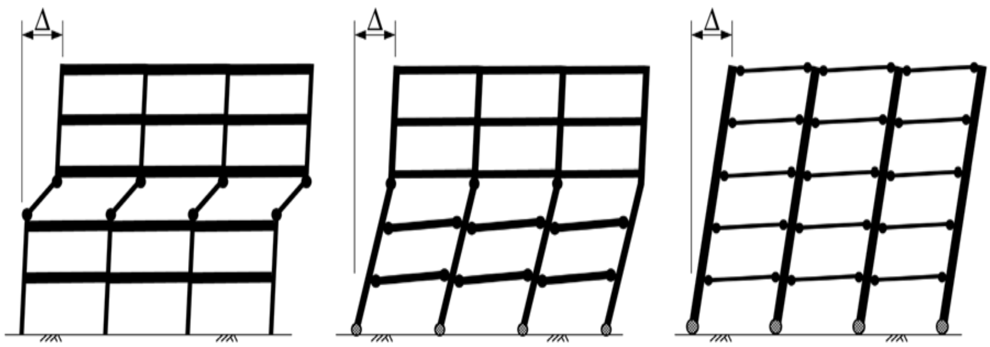
V. Gioncu (2000) performed the review for ductility related to seismic response of framed structures. The required ductility was determined at the level of full structure behaviour, while the available ductility was obtained as local behaviour of node (joint panel, connections or member ends). The checking for ductility of columns is generally a difficult operation. For SMRF structures, the column sections are enlarged to achieve a global mechanism. This over-strength of the column may reduce the available ductility of columns. At the middle frame height a drastic reduction of available ductility was observed. Since the required ductility is maximum at this height, the collapse of the building may occur due to lack of sufficient ductility. This was commonly observed during the Kobe earthquake, where many building were damaged on the storeys situated at the middle height of structure. It was observed that the factors regarding seismic actions, such as velocity and cycling loading, reduce the available ductility.

Sungjin *et al*. (2004) studied different factors affecting ductility. Evaluation of the distortion capacity of RC columns is very important in performance-based seismic design. The deformation capacity of columns is generally being expressed in numerous ways which are curvature ductility, displacement ductility or drift. The influence of concrete strength, longitudinal reinforcement ratio, volumetric ratio of confining reinforcement, shear span-to-depth ratio and axial load on various ductility factors were evaluated and discussed.

Saatcioglu & Razvi (1992) suggested that there is a direct relationship between lateral drift and concrete confinement grounded on their investigations. They resolved that the shear span to depth ratio (*L*/*h*) did not show a noticeable effect on drift capacity when the *P*-delta effect was taken into account and that the quantity of longitudinal reinforcement had an insignificant influence. They also illustrated that a rise in the concrete strength leads to reduced displacement ductility and drift capacities for a specified curvature ductility. To attain the same level of displacement ductility or drift capacity in a high strength concrete column, the usage of a greater amount of confining reinforcement was mandatory. As the quantity of longitudinal reinforcement amplified, the lateral load carrying ability, the yield displacement and the ultimate displacement capacity increased. However, the increase in the yield displacement was more distinct than the upsurge in the ultimate displacement capacity.

Moehle *et al.* (2008), conducted study on the principles of seismic design of reinforced concrete Special Moment Framesas per ACI 318. The proportioning and detailing requirements for special moment frames were provided to ensure that inelastic response is ductile. The major principles were to achieve a strong-column/weak-beam design that distributes the inelastic response over several storeys, to prevent shear failure and to provide details that enable ductile flexural response in yielding regions. When a building sways during an earthquake, the distribution of damage over height depends on the distribution of lateral drift. If the building has weak columns, drift tends to concentrate in one or a few stories (Fig: 2.1 a), and may exceed the drift capacity of the columns. On the other hand, if columns provide a stiff and strong spine over the building height, drift will be more uniformly distributed (Fig: 2.1 c), and localized damage will be reduced. It is important to recognize that the columns in a given story support the weight of the entire building above those columns, whereas the beams only support the gravity loads of the floor of which they form a part; therefore, failure of a column is of greater consequence than failure of abeam. Recognizing this behavior, building codes specify that columns be stronger than the beams that frame into them. Studies (Kuntz and Browning, 2003) have shown that the full structural mechanism of Fig: 2.1 can only be achieved if the column-to-beam strength ratio is relatively large (on the order of four). As this is impractical in most cases, a lower strength ratio of 1.2

is adopted by ACI 318. Thus, some column yielding associated with an intermediate mechanism (Fig: 2.1 b) is to be expected, and columns must be detailed accordingly.



(a) (b) (c)

Fig: 2.1: Story mechanism Intermediate mechanism Beam mechanism
(Reference: Moehle *et al*., 2008)

Beams in SMRF structures must have transverse reinforcement in the form of either hoops or stirrups throughout the length. Hoops must fully enfold the beam cross section and are provided to confine the concrete, prevent buckling of longitudinal bar, improve bond between reinforcing bars and concrete, and prevent shear failure. Stirrups are generally used where only shear resistance is required. Beams of special moment frames can be divided into three different zones when considering where hoops and stirrups can be placed: the zone at each end of the beam where flexural yielding is expected to occur; the zone along lap-spliced bars, if any; and the remaining length of the beam. The zone at each end, of length *2h*, needs to be well confined because this is where the beam is expected to undergo flexural yielding and this is the location with the highest shear. Therefore, closely spaced, closed hoops are required in this zone, as shown in Fig: 2.2. Note that if flexural yielding is expected anywhere along the beam span other than the end of the beam, hoops must also extend *2h* on both sides of that yielding location. This latter condition is one associated with non-reversing beam plastic hinges and is not recommended. Subsequent discussion assumes that this type of behaviour is avoided by design. Hoop reinforcement may be constructed of one or more closed hoops. Alternatively, it may be constructed of typical beam stirrups with seismic hooks at each end closed off with crossties having 135° and 90° hooks at opposite ends. Using beam stirrups with crossties rather than closed hoops is often preferred for constructability so that the top longitudinal beam reinforcement can be placed in the field, followed by installation of the crossties.

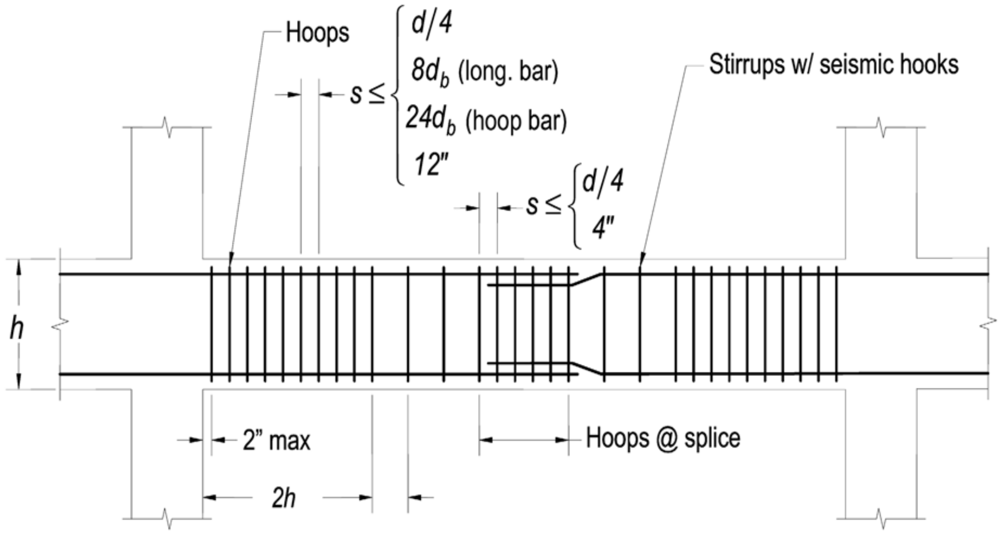


Fig: 2.2 Hoop and stirrup location and spacing requirements.

2.4 CONFINEMENT MODELS

Strain capacity of RC sections can be enhanced many folds by confining the concrete with reinforcing spirals or closed hoops. The hoops act to restrain dilation of the core concrete as it is loaded in compression, and this confinement helps in enhancement of strength and strain capacity. At low levels of stress, the behaviour of confined core concrete is similar to that of unconfined concrete. As the stress increases, the core concrete expands against the transverse reinforcement which results in a confining action in concrete. This increase of strength and ductility of core concrete by proper confinement of transverse reinforcement is an important design consideration of structural RC members in areas prone to seismic activity. Various models has been proposed for the stress-strain relation of confined concrete. The more accurate the stress-strain model, the more consistent is the assessment of strength and deformation behaviour of concrete members. An extensive review of the various existing confinement models is given below.

In Kent and Park (1971) model of stress-strain relations it was expected that concrete can tolerate some stress at indeterminately large strains. In this model the strength enhancement factor due to confinement was not considered. It was suggested that the collapse of the member would happen before the strains in concrete become unfeasibly high. Hence, for this model it was taken that the concrete can take up to a stress of 20% of peak stress.

Scott *et al.* (1982) conducted experiments on a number of square concrete columns reinforced with either 8 or 12 longitudinal bars and transversely reinforced with overlapping hoop sets. They conducted tests at rapid strain rates, distinctive of seismic loading. Unlike the Kent and Park (1971) model which was standardized against small gauge tests, they found substantial strength improvement due to the presence of good confining reinforcement details. Thus simple modifications were made to the Kent and Park (1971) model in order to incorporate the increase in the compressive strength of confined concrete at high strain rates (Fig: 2.3).The strength enhancement factor K is expressed in terms of volumetric ratio of confining reinforcement. The maximum concrete stress attained is assumed to be Kf_c' and the strain at maximum concrete stress is$0.002K$. This model of stress strain is called Modified Kent and Park model in this study.

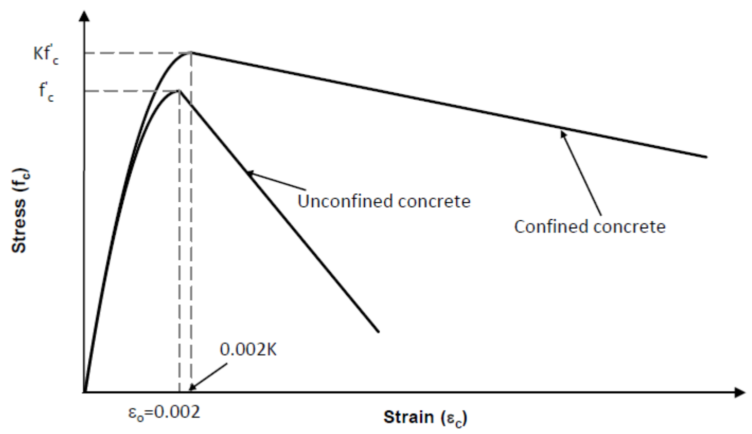


Fig: 2.3 Stress-strain behaviour of compressed concrete confined by rectangular steel ties- Modified Kent and Park (Scott *et al*. 1982) model.

Mander *et al.* (1988a) first tested circular, rectangular and square full scale columns at seismic strain rates to investigate the impact of diverse transverse reinforcement arrangements on the confinement efficacy and overall performance. Mander *et al.* (1988b) went on to model their experimental results. It was detected that if the peak strain and stress coordinates might be found ($\varepsilon_{cc}, f'_{cc}$), then the performance over the complete stress-strain range was alike, irrespective of the arrangement of the confinement reinforcement used. Thus they accepted a failure criteria based on a 5-parameter model of William and Warnke (1975) laterally with data from Schickert and Winkler (1979) to produce a comprehensive multi-axial confinement model. Then to designate the entire stress-strain curve they implemented the 3-parameter equation suggested by Popovics (1973). Due to its generality, the Mander *et al.* (1988b) model is used widespread in design and research. In this study this model is termed as Mander's Model. Typical Mander's Model stress strain curve for confined and unconfined concrete is shown in Fig: 2.4

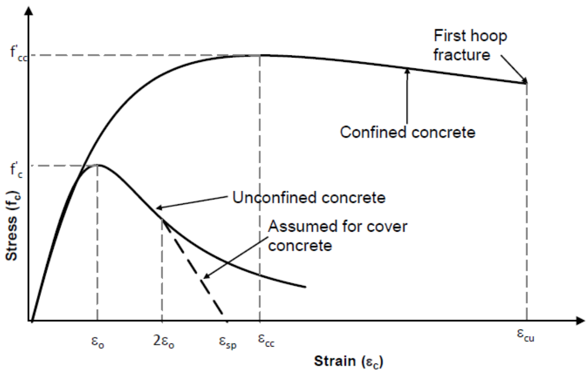


Fig: 2.4 Stress-strain relation for confined and unconfined concrete – Mander *et al.* (1988b).

Saatcioglu and Razvi (1992) proposed an analytical model to build a stress-strain connection for confined concrete. The model entails of a parabolic ascending segment, followed by a linear descending part. It was founded on computation of lateral confinement pressure produced by circular and rectilinear reinforcement, and the consequential improvements in strength and ductility of confined concrete. Confined concrete strength and corresponding strain were conveyed in terms of equivalent uniform confinement pressure delivered by the reinforcement enclosure. The descending part was calculated by defining the strain corresponding to 85% of the peak stress. This strain level is stated in terms of confinement parameters. A constant residual strength was expected beyond the descending branch, at 20% strength intensity. Stress-strain curve obtained using this model is given in Fig: 2.5. The name Razvi model is used for this particular stress strain relation throughout this study.

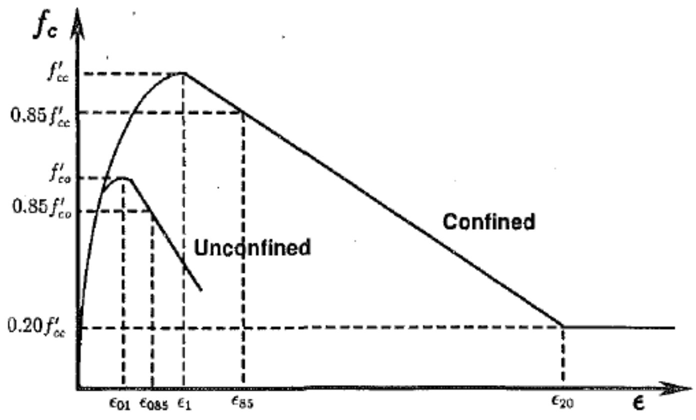


Fig: 2.5 Stress-strain relation – Saatcioglu and Razvi model(1992)

2.5 RESPONSE REDUCTION FACTOR

Mondal *et al.* (2013) conducted a study to find *R* for reinforced concrete regular frame assemblies designed and detailed as per Indian standards IS 456, IS 1893 and IS13920. Most seismic design codes today comprise the nonlinear response of a structure obliquely through a 'response reduction/modification factor' (*R*). This factor permits a designer to use a linear elastic force-based design while accounting for nonlinear behaviour and deformation limits. This research was aimed on the estimation of the actual values of this factor for RC moment frame buildings designed and detailed as per Indian standards for seismic and RC designs and for ductile detailing, and comparing these values with the value given in the design code. Values of *R* were found for four designs at the two performance levels. The results showed that the Indian standard suggests a higher value of *R*, which is potentially hazardous. Since Indian standard IS 1893 does not provide any clear definition of limit state, the Structural Stability performance level of ATC-40 was used here, both at the structure level and at the member levels. In addition to this, actual member plastic rotation capacities, were also calculated. Priestley recommended an ultimate concrete compression strain for unconfined concrete = 0.005. The ultimate compressive strain of concrete confined by transverse reinforcements as defined in ATC-40 was taken in this work to obtain the moment characteristics of plastic hinge segments. In order to prevent the buckling of longitudinal reinforcement bars in between two successive transverse reinforcement hoops, the limiting value of ultimate strain was limited to 0.02. Suitable modelling of the preliminary stiffness of RC beams and columns is one of the important aspects in the performance evaluation of reinforced concrete frames. Two performance limits (PL1 and PL2) were considered for the estimation of R for the study frames. The first one resembled to the Structural Stability limit state defined in ATC-40. This limit state is well-defined both at the storey level and at the member level. The second limit state was based on plastic hinge rotation capacities that were found for each individual member depending on its cross-section geometry. The global performance limit for PL1 was demarcated by a maximum inter-storey drift ratio of 0.33Vi/Pi. The *R* values attained were ranging from 4.23 to 4.96 for the four frames that were considered, and were all lesser than specified value of *R* (= 5.0) for SMRF frames in the IS 1893. The taller frames exhibited lower *R* values. Component wise, the shorter frames (two-storey and four-storey) had more over-strength and *Rs*, but slightly less ductility and R_μ compared to the taller frames. According to Performance Limit 1 (ATC-40 limits on inter-storey drift ratio and member rotation capacity), it was found that the Indian standard overestimates the *R* factor, which leads to the potentially dangerous underestimation of the design base shear. Based on Performance Limit 2 the IS 1893 recommendation was found to be on the conservative side.

Krawinkler *et al*. (1998) studied the advantages and disadvantages of Pushover analysis and suggested that element behaviour cannot be assessed in the state of currently employed global system quality factors such as the *R* and R_w factors used in existing US seismic codes. They also recommended that pushover analysis will deliver insight into structural aspects that control performance during severe earthquakes. For structures that vibrate chiefly in the fundamental mode, the pushover analysis will very probably provide good estimations of global, as well as local inelastic, deformation demands. This analysis will also expose design weaknesses that may remain hidden in an elastic analysis. Such weaknesses include story mechanisms, excessive deformation demands, strength irregularities and overloads on potentially brittle elements such as columns and connections.

Asgarian and Shokrgozar (2008) evaluated over-strength, ductility and response modification factor of Buckling Restrained Braced frames. Seismic codes consider a decrease in design loads, taking benefit of the fact that the structures possess substantial reserve strength (over-strength) and capacity to disperse energy (ductility). The over-strength and the ductility are incorporated in structural design through a force reduction or a response modification factor. This factor represents ratio of maximum seismic force on a structure through specified ground motion if it was to remain elastic to the design seismic force. Thus, seismic forces are reduced by the factor *R* to obtain design forces. The basic fault in code actions is that they use linear methods not considering nonlinear behaviour. The structure can engross quiet a lot of earthquake energy and repels when it enters the inelastic zone of deformation. Over-strength in structures is connected to the fact that the maximum lateral strength of a structure usually beats its design strength. It was perceived that the response modification factor drops as the height of building increases. This result was outward in all type of bracing outline.

Mendis *et al.* (1998) reviewed the traditional force-based (FB) seismic design method and the newly proposed displacement-based (DB) seismic assessment approach. A case study was done for reinforced concrete (RC) moment-resisting frames designed and detailed according to European and Australian earthquake code provisions, having low, medium and high ductility capacity. Response reduction factor (*R)* for Ordinary Moment Resisting frame is ‘4’ as per AS 3600 while for Special Moment Resisting frame, *R*= 8 as per ACI 318–95. It was observed that OMRF developed plastic hinges in the columns under the El Centro earthquake and SMRF generally developed plastic hinges in the beams rather than the columns. This was consistent with the ACI 318–95 strong column-weak beam detailing philosophy used in the design of this SMRF. The displacement ductility and rotation ductility demands of the SMRF during the El Centro earthquake were some 3 times that of the OMRF.

2.6 PUSH-OVER

Jianguo *et al.* (2006), investigated the seismic behavior of concrete-filled rectangular steel tube structures. A push-over analysis of a 10-story moment resisting frame (MRF) composed of CFRT columns and steel beams were conducted. The results show that push-over analysis is sensitive to the lateral load patterns, so the use of at least two load patterns that are expected to bound the inertia force distributions was recommended.

Push-over analysis was found useful in estimating the following characteristics of a structure: 1) the capacity of the structure as represented by the base shear versus top displacement graph; 2) the maximum rotation and ductility of critical members; 3) the distribution of plastic hinges at the ultimate load; and 4) the distribution of damage in the structure, as expressed in the form of local damage indices at the ultimate load. In frame structures plastic hinges usually form at the ends of beams and columns under earthquake action. For beam elements, plastic hinges are mostly caused by uniaxial bending moments, whereas for column elements, plastic hinges are mostly caused by axial loads and biaxial bending moments. Therefore it was concluded that, in push-over analysis different types of plastic hinges should be applied for the beam elements and the column elements separately.

Chugh (2004) explained the validity of non-linear analysis for seismic design of structures. He suggested that

- The linear performance is restricted to the area of small response.
- When the stresses are excessive, material nonlinearity reveals.
- When the displacements are large, geometric nonlinearity manifests.

If the loading is removed in the large response domain, there will be a residual response. Once yielding takes place (at any section), the behaviour of a statically indeterminate structure enters an inelastic phase, and linear elastic structural analysis is no longer valid. It would be too expensive to design a structure based on the elastic spectrum, and the code (IS 1893) allows the use of a response reduction factor (R), to reduce the seismic loads. But this reduction will be possible, without collapse of the structure, provided sufficient ductility is in-built through proper design of the structural elements. To get a correct response, we must resort to non-linear analysis. This is also called limit analysis.

Sadjadi *et al* (2006) proposed a nonlinear static analysis, also acknowledged as a push-over analysis, which involves laterally pushing of the structure in one direction with a certain lateral force or displacement distribution until either a specified drift is attained or a numerical instability has occurred. Push-over analysis is an effective way to study the behaviour of the assembly, emphasizing the order of member cracking and yielding as the base shear value increases. This information then can be used for the estimation of the performance of the structure and the sites with inelastic deformation of over-strength and to get a sense of the general capacity of the structure to withstand inelastic deformation. Pushover analysis finds the locations that are expected to be endangered to large inelastic deformations, which helps in the evaluation of the performance of the structure, and design of component detailing. As was mentioned earlier, the pushover inter-storey drift distributions are basically first mode while the dynamic inter-storey drift distributions contain substantial second mode influences. This implies that the static pushover examination for irregular structures cannot be accurate**.**

Bansal (2011) preferred Pushover analysis as the method for seismic performance study of structures by the major restoration guidelines and codes as it is theoretically and computationally easy. Pushover analysis allows drawing the order of yielding and failure on element and structural level as well as the development of overall capacity curve of the arrangement. It is a method by which a computer model of the building is exposed to a lateral load of a certain shape. The intensity of the lateral load is gradually increased and the sequence of cracks, yielding, plastic hinge formation, and failure of various structural components is recorded.

Mehmet *et al*. (2006), explained that due the easiness of Pushover analysis, the structural engineers have been using the nonlinear static method or pushover analysis. Pushover analysis is performed for various nonlinear hinge characters available in certain programs based on the FEMA-356 and ATC-40 guidelines and he pointed out that Plastic hinge length has significant effects on the displacement capacity of the structures. The alignment and the axial load degree of the columns cannot be considered properly by the default-hinge properties.

Shuraim *et al*. (2007) utilized the nonlinear static analytical procedure (Pushover) as introduced by ATC-40 for the estimation of existing design of a fresh reinforced concrete frame. Possible structural shortages in reinforced concrete frame, when exposed to a moderate seismic loading, were assessed by the pushover tactics. In this method the design was valued by redesigning under nominated seismic blend in order to show which elements would require added reinforcement. Most columns demanded significant additional reinforcement, signifying their weakness when subjected to seismic forces. The nonlinear pushover procedure displays that the frame is adept of enduring the reputed seismic force with some significant yielding at all beams and one column.

Kadid and Boumrkik (2008), proposed use of Pushover Analysis as a feasible method to judge damage liability of a building designed rendering to Algerian code. Pushover analysis was a Series of incremental static analysis carried out to improve a capacity curve for the structure. Based on capacity curve, a target displacement which was an estimate of the displacement that the design earthquake would produce on the building was obtained. The extent of damage suffered by the structure at this target displacement is counted representative of the Damage experienced by the structure when subjected to design level ground shaking. Since the behaviour of reinforced concrete structures could be highly inelastic when subjected to seismic loads, the total inelastic performance of RC constructions would be conquered by plastic yielding effects and consequently the exactness of the pushover analysis would be affected by the ability of the analytical models to arrest these effects

Khose *et al*. (2012), conducted a case study of seismic performance of a ductile RC frame building designed using four major codes, ASCE7, EN1998,NZS 1170 and IS 1893 . The performance of the test building was evaluated using the Displacement Modification Method (DMM) as well as the guidelines of ASCE-41. The variation in capacity curves is a result of combined effect of the differences in design spectra, effective member stiffness, response reduction factors, load and material factors, as well as load combinations. The buildings designed for other codes (New Zealand and Euro-code) have significantly lower strengths than the buildings of comparable ductility classes designed for Indian and American codes. In case of DBE, all the considered codes result in Life Safety (LS) or better performance levels in both the directions, except in case of Euro-code 8 in both the directions and NZS 1170.5 in transverse direction.

2.7 SUMMARY

This chapter dealt with the numerous numbers of papers and journals that has been found helpful for carrying out the work. An extensive literature review is done and the inference is noted down. It is well established from various studies that ductile detailing is necessary to resist earthquakes. SMRF buildings exhibit higher ductility and resistance to seismic loading through proper confinement of transverse reinforcement compared to OMRF buildings. A detailed review of the above models in addition to IS 456 model is done in this study. In-order to study the ductility, response reduction factors are to be calculated which can be obtained using non-linear static pushover analysis. For obtaining a much reliable pushover curve of frames, a stress-strain confinement model which actually distinguishes the behaviour of confined and unconfined concrete has to be used. From the study of literature, it has been observed that Mander's model, Razvi model and Modified Kent and Park model can be considered for the present study.

CHAPTER 3

REVIEW OF EXISTING CONFINEMENT MODELS FOR CONCRETE

3.1 GENERAL

First part of this Chapter deals with various confinement models for the stress-strain relationship of concrete. The confinement in the concrete plays a major role in the strength and ductility of the RC members. In order to show the effect of considering the confinement in the stress-strain curve and its effects in the strength and ductility, various sections specially detailed for confinement has to be designed. Hence a number of building frames are considered and designed as both Special Moment Resisting Frames (SMRF) and Ordinary Moment Resisting Frames (OMRF). The configuration of the frames and the reinforcement details of RC sections are also presented in this Chapter. Confinement stress-strain curves for various SMRF and OMRF sections are also developed as per various available models.

3.2 CONFINEMENT CHARACTERISTICS OF CONCRETE

Provision for ductility is of utmost importance in the design and detailing of RC structures subjected to seismic loads. To accomplish this, IS 13920 specifies the use of transverse reinforcement or stirrups in structural members like columns. The effects of confinement completely affect magnitude of stress- strain curve of concrete which leads to an increase in compressive force of concrete. But IS code design is completely based on the simplified stress block of unconfined concrete and it does not consider the gain in strength due to confinement. To study the effects of lateral confinement on column capacity an investigative study is carried out. The more accurate the stress-strain model, the more consistent is the assessment of strength and deformation behavior of concrete members. It is to be noted that concrete exhibits different performance in the confined and unconfined conditions. Confined concrete exhibits enhanced strength as well as greater ductility compared to unconfined concrete. This necessitates the use of a stress-strain model that distinguishes the behavior of confined and unconfined concrete. The stress-strain diagrams for concrete are developed by considering various confinement models and compared with the stress-strain diagram as per the IS 456 (2000).

3.2.1 Review of Existing Confinement Models

IS 456 (2000)

The stress- strain curve as per IS 456 assumes a parabola in the ascending branch with strain of 0.002 corresponding to peak strength and then the stress remains constant until the strain reaches an ultimate value of 0.0035. The descending branch in the post-peak region is not accounted for and the strength and ductility enhancement due to confinement is not considered. Thus IS 456 (2000) proposes the same strength and ductility for confined and unconfined concrete which may underestimate the strength and ductility of the sections and the building frame as a whole. In real case, the post-peak behavior is a descending branch, which is due to 'softening' and micro-cracking in the concrete.The stress strain relations as per IS code is given below.

For $\varepsilon_c \leq \varepsilon_{co}$ $\quad f_c = f'_{co}\left[\frac{2\varepsilon_c}{0.002} - \left(\frac{\varepsilon_c}{0.002}\right)^2\right]$ (3.1)

For $\varepsilon_{co} < \varepsilon_c < 0.0035$ $\quad f_c = f'_{co}$ (3.2)

where f_c is the stress in concrete corresponding to the strain ε_c and f'_{co} is the strength concrete corresponding to the strain 0.002 (ε_{co}).

Mander's model

Mander *et al.* (1988a) suggests that confinement reinforcement increases the ductility as well as column strength. The model incorporates a strength enhancement factor due to confinement effect. But a single equation is used for both the ascending and descending branches in this model. The stress strain curve for confined concrete approaches to that of unconfined when the confinement is negligible. It requires four coordinates to define the stress strain curve. The four coordinates are peak stress, corresponding strain, ultimate strain and corresponding stress. In many cases, the ultimate strain predicted by this model is found to be less than that of the strain corresponding to peak stress, which makes the representation incomplete. This model may require some modification due to this draw back as also pointed out by Durga *et al*. (2013). The governing equations for this stress strain model are given below.

The peak strength, $$f'_{cc} = f'_{co}\left[1 + 3.7\left(\frac{0.5k_e\rho_s f_{yh}}{f'_{co}}\right)^{0.85}\right] \quad (3.3)$$

Where f'_{co} is unconfined compressive strength equal to $0.75f_{ck}$, k_e is the confinement effectiveness coefficient having a typical value of 0.95 for circular sections and 0.75 for rectangular sections, ρ_s = Volumetric ratio of confining steel, f_{yh}= Grade of confining steel,

Strain corresponding to peak stress, $$\varepsilon_{cc} = \varepsilon_{co}\left[1 + 5\left(\frac{f'_{cc}}{f'_{co}} - 1\right)\right] \quad (3.4)$$

The ultimate compressive strain, $$\varepsilon_{cu} = 0.004 + \frac{0.6\rho_s f_{yh}\varepsilon_{sm}}{f'_{co}} \quad (3.5)$$

Where ε_{sm}= Steel strain at maximum tensile stress,

The stress at any strain, $$f_c = \frac{f'_{cc}xr}{r-1+x^r} \quad (3.6)$$

Where, $x = \frac{\varepsilon_c}{\varepsilon_{cc}},\ r = \frac{E}{E_c - E_{sec}},\ E_c = 5000\sqrt{f'_{co}},\ E_{sec} = \frac{f'_{cc}}{\varepsilon_{cc}}$ (3.7)

Kent-Scott-Park Model - Modified Kent and Park Model (1982)

Strength enhancement factor in this model depends on the ratio of volume of confining reinforcement to the volume of the confined core concrete and also on the unconfined compressive strength of concrete. In this model the ascending and descending branches are characterised by different equations.

The peak strength, $$f'_{cc} = Kf'_c \quad (3.8)$$

$$K = 1 + \frac{\rho_s f_{yh}}{f'_c} \quad (3.9)$$

$$\rho_s = \frac{2(b'' + d'')A''_s}{b''d''s} \quad (3.10)$$

ρ_s = Volumetric ratio of confining steel, f_{yh}= Grade of confining steel

s = vertical spacing of hoops measured centre to centre , A''_s is the area of the transverse reinforcement, b'' and d'' are the core dimensions measured outer to outer of ties in x and y directions respectively.

Strain corresponding to peak stress, $\varepsilon_0 = 0.002K$ (3.11)

For $\varepsilon_c \leq 0.002K$ $\quad f_c = Kf'_c\left[\frac{2\varepsilon_c}{0.002K} - \left(\frac{\varepsilon_c}{0.002K}\right)^2\right]$ (3.12)

For $\varepsilon_c > 0.002K$ $\quad f_c = Kf'_c[1 - Z_m(\varepsilon_c - 0.002K)] < 0.2Kf'_c$ (3.13)

$$Z_m = \frac{0.5}{\varepsilon_{50h} + \varepsilon_{50u} - \varepsilon_0} \quad (3.14)$$

$$\varepsilon_{50h} = \frac{3}{4}\rho_s\sqrt{\frac{b''}{s}} \quad (3.15)$$

$$\varepsilon_{50u} = \frac{3 + 0.29f'_c}{145f'_c - 1000} \quad (f'_c\ in\ MPa) \quad (3.16)$$

Saatcioglu and Razvi Model (1992)

The stress-strain curve of confined concrete is characterised by a parabolic ascending portion then continues with a linear descending branch and a constant residual strength at 20% of peak stress. In this model the peak strength of confined concrete is expressed as a function of lateral confining pressure. As the confining pressure increases, the strength as well as the ductility of the confined concrete increases.

The peak strength, $\quad f_{cc} = K_3 f_{ck} + K_1\sigma_{2e}$ (3.17)

Where $K_1 = \frac{6.7}{(\sigma_{2e})^{0.17}}, K_3 = 0.85$ (3.18)

The strain corresponding to the peak stress f_{cc}, $\varepsilon_{coc} = \varepsilon_{co}[1 + 5\lambda]$ (3.19)

Where ε_{co} = 0.002, $\quad \lambda = \frac{K_1\sigma_{2e}}{K_3 f_{ck}}$ (3.20)

For a square section σ_{2e} is expressed as, $\sigma_{2e} = \beta\sigma_2$ (3.21)

where;

$$\sigma_2 = \frac{\Sigma A_0 f_{ywk}(\sin\alpha)}{s * b_k} \quad (3.22)$$

$$\beta = 0.26\sqrt{\left(\frac{b_k}{a}\right)\left(\frac{b_k}{s}\right)\left(\frac{1}{\sigma_2}\right)} \quad (3.23)$$

$s =$ vertical spacing of hoops measured centre to centre , b_{kx} and b_{ky} are the core dimensions measured centre to centre of ties in x and y directions respectively.

Ascending portion:

$$\sigma_c = f_{cc}\left[\left(\frac{2\varepsilon_c}{\varepsilon_{coc}}\right) - \left(\frac{\varepsilon_c}{\varepsilon_{coc}}\right)^2\right]^{\frac{1}{1+2\lambda}} \leq f_{cc} \quad (3.24)$$

Descending portion:

$$\varepsilon_{c85} = 260\rho\varepsilon_{coc} + \varepsilon_{u85} \quad (3.25)$$

ε_{u85}=0.0038 (for unconfined concrete) (3.26)

$$\rho = \frac{\Sigma A_{oxy} \sin\alpha}{s(b_{kx}+b_{ky})} \quad (3.27)$$

b_{kx} and b_{ky} are the core dimensions measured centre to centre of ties in x and y directions respectively. ΣA_{oxy} is the summation of cross-sectional areas of the ties on sections taken in x and y directions

The confinement models considered in the present study is summarised above. The comparison between various models requires RC sections designed as SMRF and OMRF. Following section explains the details of building frames.

3.3 BUILDING CONFIGURATIONS AND DESIGN DETAILS

A total of 4 plane frames are selected with number of storeys 2, 4, 8 and 12, keeping the same number of bays as shown in Fig 3.1. The storey height and bay width of all the frames are 3 m and 5 m respectively. The frames are assumed to be located in seismic zone IV, the soil type chosen is medium and importance factor of 1.0 is assumed. The dead and live loads are calculated using IS 875 Part 1 (1987) and lateral loads are calculated as per IS 1893(2002).

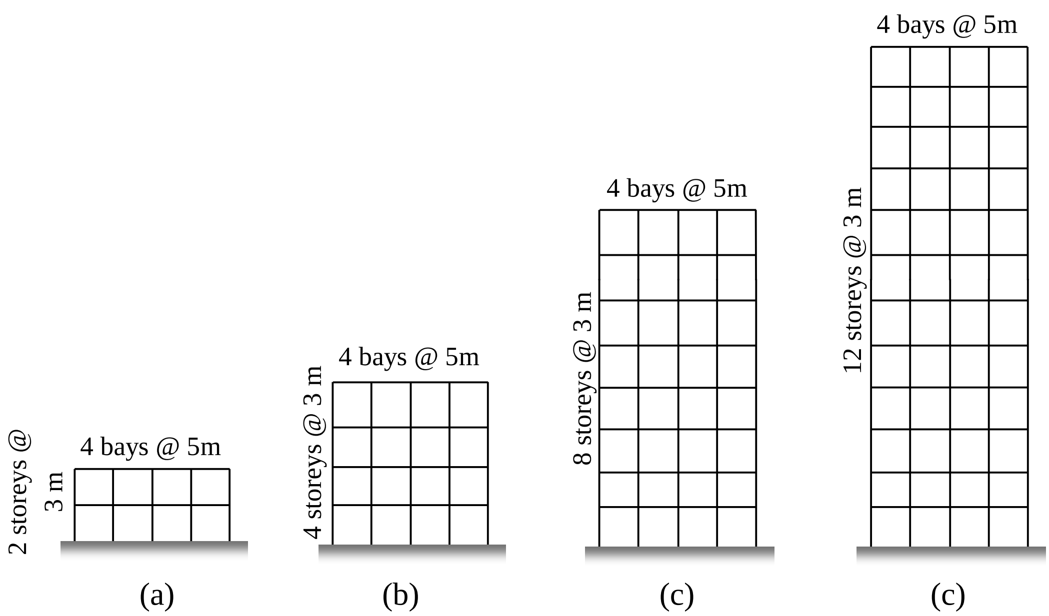


Fig 3.1 Elevation of frames considered

Each plane frame is designed as both SMRF and OMRF. OMRF frames are designed with a response reduction factor of 3 and SMRF with a response reduction factor of 5 in compliance with IS 1893 (2002). The design of RC sections are done as per IS 456 for OMRF frames and the design and ductile detailing of SMRF frames are done conforming to IS 13920 specifications. For convenient and easy presentation of frames, a naming standard has been used. The frame designated as 4S4B-SMRF represents SMRF building with four storeys and four bays. The designation, type of design, R factor and analysis, design and detailing provisions followed are tabulated in the Table 3.1.

Table 3.1: Details of the Moment Resisting Frames considered

Sl No:	Frame Tag	No. of storey	No. of bays	Frame type	R	Analysis Design & Detailing
1	2S4B- SMRF	2	4	SMRF	5	IS 1893 & IS 13920
2	2S4B- OMRF	2	4	OMRF	3	IS 1893 & IS 456
3	4S4B- SMRF	4	4	SMRF	5	IS 1893 & IS 13920
4	4S4B- OMRF	4	4	OMRF	3	IS 1893 & IS 456
5	8S4B- SMRF	8	4	SMRF	5	IS 1893 & IS 13920
6	8S4B- OMRF	8	4	OMRF	3	IS 1893 & IS 456
7	12S4B- SMRF	12	4	SMRF	5	IS 1893 & IS 13920
8	12S4B- OMRF	12	4	OMRF	3	IS 1893 & IS 456

Moment resisting frame structures of different heights are selected to characteristically represent short, medium and long period structures.In the present study, the grade of steel used is Fe 415.compressive strength of the cube (fck)is considered as 25 MPawhich corresponds to a cylinder strength (fc')of 21.25 MPa. The modulus of elasticity of steel considered is 200 GPa and that of concrete is 25 GPa ($5000\sqrt{f_{ck}}$). The live load is taken as 3 kN/m^2. The unit weight of concrete and brick masonry infill is taken as 25 kN/m^3 and 19 kN/m^3 (including the floor finishes) respectively. The thickness of slab is assumed as 175 mm and that of infill wall is taken as 230 mm. The reinforcement details for the RC sections are given in Table 3.4 and Table 3.5. A naming convention has been done for the RC sections used in frames as shown in Table 3.4 and Table 3.5. A section designated as 450C-4S4B-SM indicates a column section of size 450 x 450 in the four storey four bay SMRF frame. Similarly, for a section designated as 350B-2S4B-SM indicates a beam section of depth 350 mm in the two storey four bay SMRF frame. The value of the various factors considered for the estimation of design horizontal seismic co-efficient,A_h is given in Table 3.2 and Table 3.3.

Table 3.2: Response Spectrum Factors Considered for the Frames

Factors	**SMRF**	**OMRF**
Seismic Zone	IV	IV
Zone Factor	0.24	0.24
Type of Building, Z	Regular office Building	Regular office Building
Importance Factor, I	1	1
Response Reduction Factor, R	5	3
Type of Soil	Medium	Medium
Damping	5%	5%

Table 3.3: Details of time periods, seismic weight and design base shear

Frame Type	**Height (m)**	**Time Period, T (sec)**	S_a/g	A_h	**Seismic Weight, W (kN)**	**Design Base Shear, V_d(kN)**
2S4B-SMRF	6.0	0.2875	2.5	0.06	3537.3	212
2S4B-OMRF	6.0	0.2875	2.5	0.1	3804.7	380.4
4S4B-SMRF	12.0	0.483	2.5	0.06	5356.11	321.36
4S4B-OMRF	12.0	0.483	2.5	0.1	5408.9	540.89
8S4B-SMRF	24.0	0.813	1.672	0.04	10790.02	431.613
8S4B-OMRF	24.0	0.813	1.672	0.0668	11156.45	745.25
12S4B-SMRF	36.0	1.1022	1.233	0.0295	17146.31	505.87
12S4B-OMRF	36.0	1.1022	1.233	0.0493	17649.81	853.035

It can be noted from Table 3.3 that as the height of the building increases, the time period also increases and the spectral acceleration co-efficient , S_a/g decreases. This variation of S_a/g and time period with number of storeys is shown in Fig: 3.2.

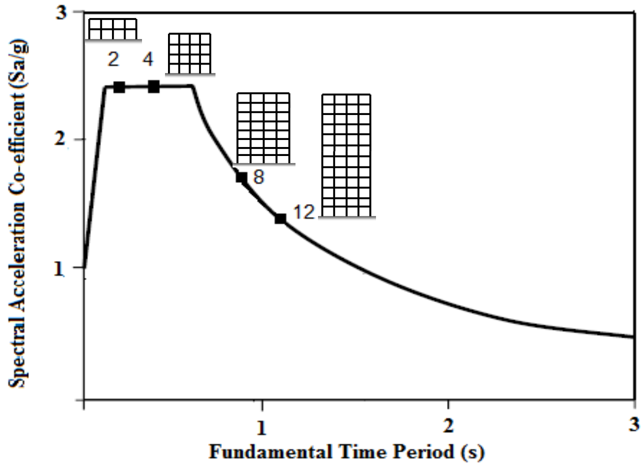


Fig: 3.2: Variation in Time Period and Spectral Acceleration Co-efficient with number of storeys

Table 3.4: Reinforcement Details for Columns

Section Tag	**Building Configuration**	**Section Size (mm x mm)**	**Longitudinal Reinforcement**	**Shear Reinforcement**
400C-2S4B-SM	2S4B-SMRF	400 x 400	8 # 16 mm	2 legged 10mm @ 85mm c/c
450C-2S4B-OM	2S4B-OMRF	450 x 450	4 # 25 mm	2 legged 8mm @ 230mm c/c
450C-4S4B-SM	4S4B-SMRF	450 x 450	4 # 25 mm	2 legged 12mm @ 85mm c/c
500C-4S4B-OM	4S4B-OMRF	500 x 500	8 # 20 mm	2 legged 8mm @ 190mm c/c
550C-8S4B-SM	8S4B-SMRF	550 x 550	8 # 20 mm	2 legged 12mm @ 75mm c/c
650C-8S4B-0M	8S4B-OMRF	650 x 650	8 # 25 mm	2 legged 8mm @ 190mm c/c
600C-12S4B- SM	12S4B-SMRF	600 x 600	12 # 20 mm	2 legged 10mm @ 75mm c/c
700C-12S4B- OM	12S4B-OMRF	700 x 700	8 # 25 mm	2 legged 8mm @ 190mm c/c

Table 3.5: Reinforcement Details for Beams

Section Tag	Building Configuration	Section Size (mm x mm)	Longitudinal Reinforcement		Shear Reinforcement
			Top	Bottom	
350B-2S4B- SM	2S4B-SMRF	300 x 350	7 # 20 mm	5 # 16 mm	2 legged 10mm @ 100mm c/c
350B-2S4B- OM	2S4B-OMRF	300 x 350	8 # 20 mm	5 # 16 mm	2 legged 8mm @ 230mm c/c
375B-4S4B- SM	4S4B-SMRF	300 x 375	6 # 20 mm	2 # 20 mm	2 legged 10mm @ 100mm c/c
375B-4S4B- OM	4S4B-OMRF	300 x 375	6 # 20 mm	3 # 20 mm	2 legged 8mm @ 230mm c/c
400B-8S4B- SM	8S4B-SMRF	300 x 400	6 # 20 mm	3 # 20 mm	2 legged 10mm @ 100mm c/c
400B-8S4B- OM	8S4B-OMRF	300 x 400	5 # 25 mm	8 # 12 mm	2 legged 8mm @ 230mm c/c
600B-12S4B- SM	12S4B-SMRF	300 x 600	6 # 20 mm	10 # 12 mm	2 legged 10mm @ 100mm c/c
600B-12S4B-OM	12S4B-OMRF	300 x 600	5 # 25 mm	10 # 12 mm	2 legged 8mm @ 230mm c/c

3.4 COMPARISON OF STRESS-STRAIN CURVES FOR THE DESIGNED SECTIONS

The stress-strain curve of concrete depends on the amount of confinement. In order to show the comparison of stress-strain curve using various models, the RC sections of the building frames discussed in the previous section are considered.

The parameter for strength enhancement as per the two confinement models are calculated for each sections and tabulated in the table 3.6. The values of stress strain data are calculated using the strength enhancement parameter as per various confinement models discussed in the above section for selected RC sections. The obtained stress-strain curves are plotted in the Fig 3.3, Fig 3.4, Fig 3.5 and Fig 3.6.

Table 3.6: Confinement Factors for Column Sections as per Kent and Park Model

Section	Column Section (mm x mm)	Hoop Volumetric Ratio (ρ_s)	Strength Enhancement Factor (K)
400C-2S4B-SM	400 x 400	0.0238	1.4654
450C-2S4B-OM	450 x 450	0.0048	1.0940
450C-4S4B-SM	450 x 450	0.0297	1.5803
500C-4S4B-OM	500 x 500	0.0051	1.1002
550C-8S4B-SM	550 x 550	0.0263	1.5141
650C-8S4B-OM	650 x 650	0.0037	1.0730
600C-12S4B-SM	600 x 600	0.0104	1.3206
700C-12S4B-OM	700 x 700	0.0034	1.0670

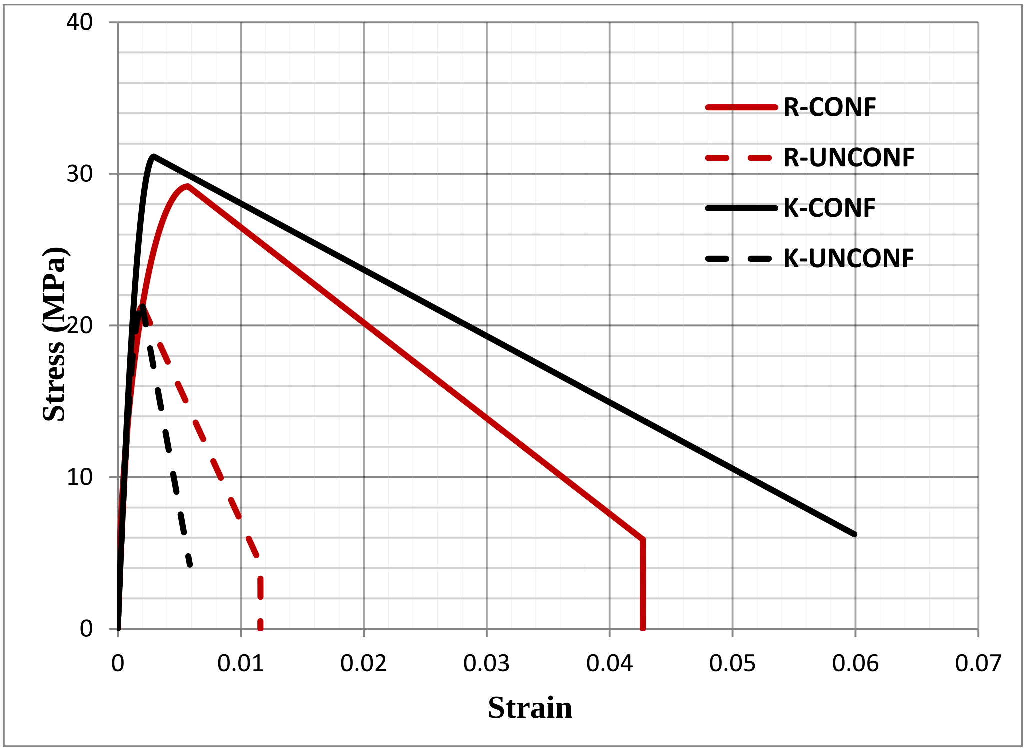


Fig: 3.3: Comparison of stress-strain curves using two confinement models (Razvi and Modified Kent models) for the RC section 400C-2S4B-SM (K_1 = 6.47, K = 1.47)

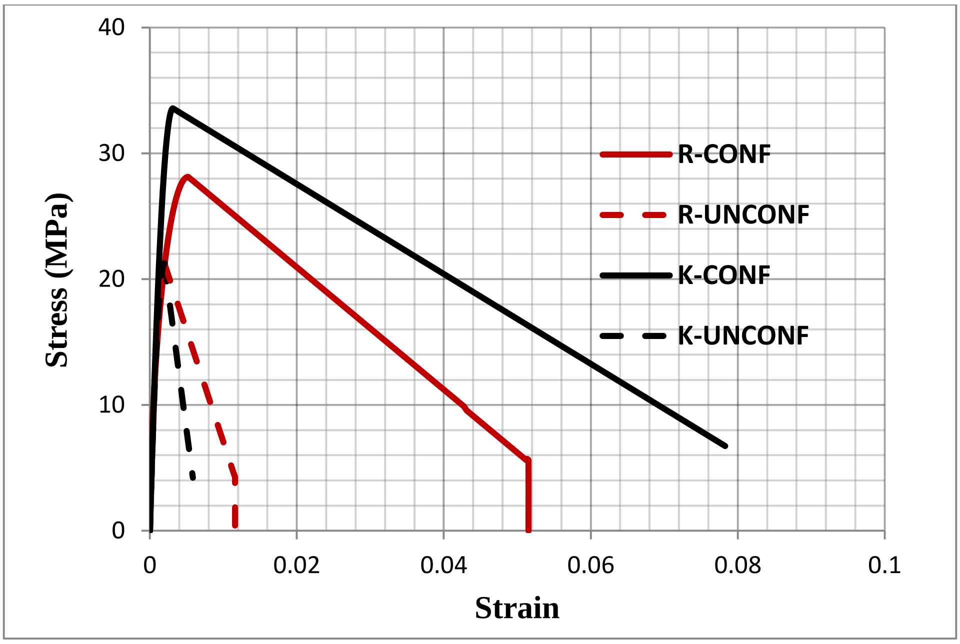


Fig: 3.4: Comparison of stress-strain curves using two confinement models (Razvi and Modified Kent models) for the RC section 450C-4S4B-SM (K_1 = 6.67, K = 1.58)

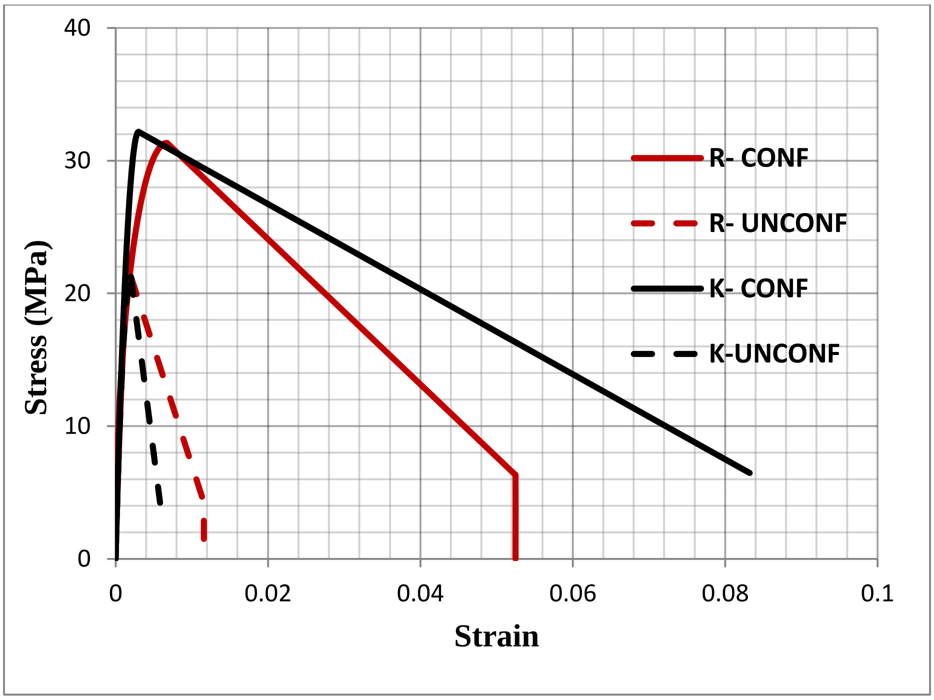


Fig: 3.5: Comparison of stress-strain curves using two confinement models (Razvi and Modified Kent models) for the RC section 550C 8S4B SM (K_1 =6.16, K = 1.51)

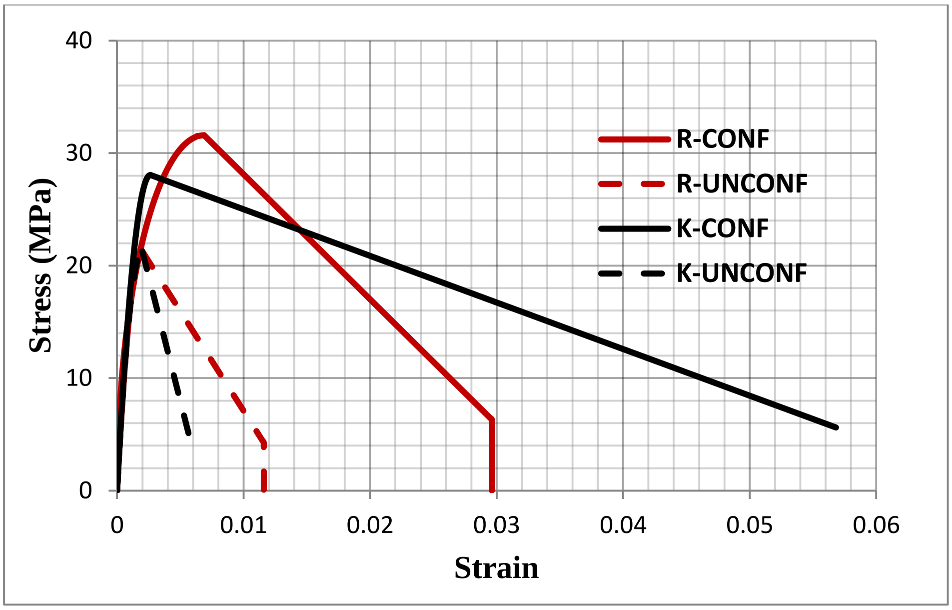


Fig: 3.6: Comparison of stress-strain curves using two confinement models (Razvi and Modified Kent models) for the RC section 600C 12S4B SM (K_1 = 6.13, K = 1.47)

3.4.1 Parametric study

A parametric study is conducted to understand the variation of stress – strain curve of concrete when the parameters such as spacing of stirrups, grade of concrete and grade of transverse steel. Fig: 3.7 shows the variation of stress-strain curve of concrete with the variation in spacing of transverse reinforcement from 75 mm to 120mm. As the spacing is decreased from 120mm to 75mm the peak strength of confined stress-strain curve is increased by 16.7% and the ultimate strain increased by about 90%.

Fig: 3.8 shows the variation of stress-strain curve of concrete with the variation in grade of transverse reinforcement from 250 MPa to 500 MPa. As the grade of steel is increased from 250MPa to 500MPa the peak strength of the confined stress-strain curve is increased by 22.6% while the ultimate strain remained same.

Fig: 3.9 shows the variation of stress-strain curve of concrete with the variation in grade of concrete from 15 MPa to 30 MPa. As the grade of concrete is increased from 15 MPa to 30 MPa the peak strength is increased by 50% and the ultimate strain decreased by 7%.

It can be seen that the ultimate strain is more dependent on the spacing of transverse reinforcement than any other parameter. Hence the spacing of stirrups shall be treated as an important factor to be ensured in the special detailing of RC sections.

Strength enhancement factor is the measure of increase in lateral confining pressure due to the transverse steel. Strength enhancement factor depends on many parameters such spacing of stirrups, grade of transverse steel, grade of unconfined concrete, dimension of confinement core. Fig: 3.10 shows the variation of stress-strain curve of concrete with the variation in strength enhancement factor (obtained value for specific cases of design) from 1.32 to 1.58. As the strength enhancement factor changes from 1.32 to 1.58 the peak strength is increased by 18.5% and the ultimate strain is increased by 46.89%.

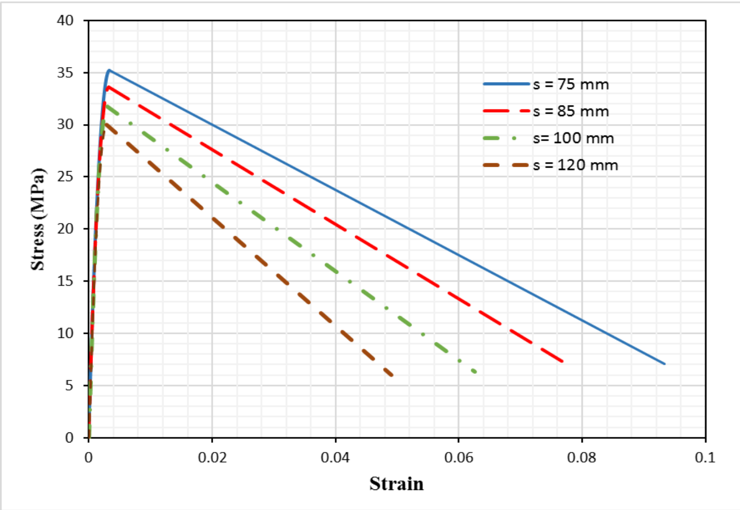


Fig: 3.7: Variation in stress-strain curve with the spacing of stirrups for the RC section 450C-4S4B-SM with the parameters, Fe415 steel and M25 concrete

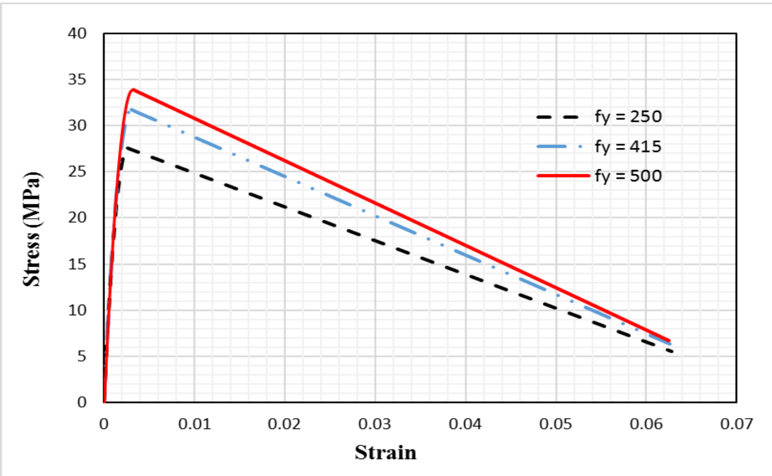


Fig: 3.8: Variation in stress-strain curve with the grade of transverse reinforcement for the RC section 450C-4S4B-SM with the parameters, spacing 100mm, and M25 concrete

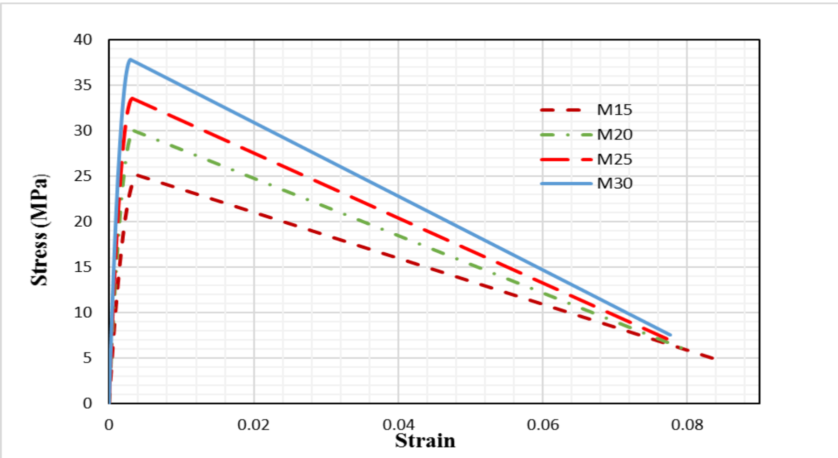


Fig: 3.9: Variation in stress-strain curve with the grade of concrete for the RC section 450C-4S4B-SM with the parameters, spacing 85mm, and Fe415 transverse steel

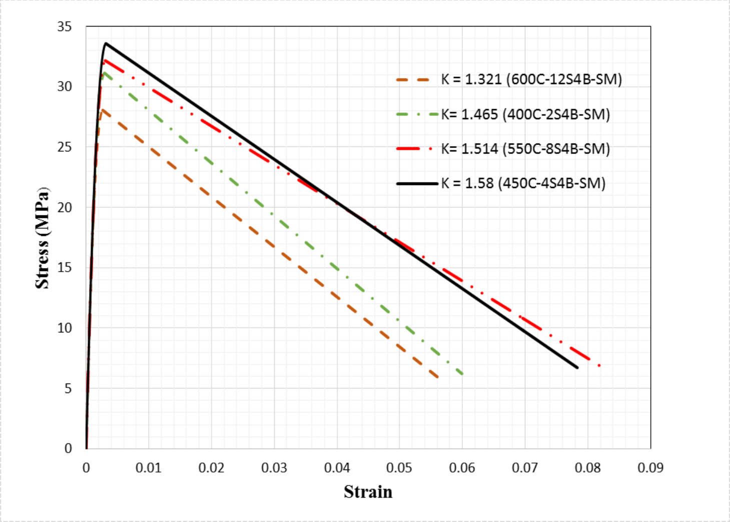


Fig: 3.10: Variation in stress-strain curve with strength enhancement factor *K*

3.4.2 Comparison of confinement models with IS 456 (2000) Model

IS 456 (2000) recommends a stress-strain curve which does not consider the effect of confinement. In order to study the difference between the stress-strain curves prescribed by IS code and modified Kent model and Razvi model, the corresponding stress-strain curves are plotted in single graph as shown in Fig: 3.11. Percentage increase in concrete strength according to Modified Kent model is about 58% while it is 32% for Razvi model compared to that of IS code.

Rajeev and Tesfamariam (2012), Alam and Kim (2012), Durga *et al.* (2013) used modified Kent and Park model for seismic response study of RC frames.

Based on the experimental study conducted by Sharma *et al.* (2009) it was concluded that response estimations using the Modified Kent and Park model closely matched the experimental results in the Indian scenario. This model is used further in the present study for the estimation of ductility parameters.

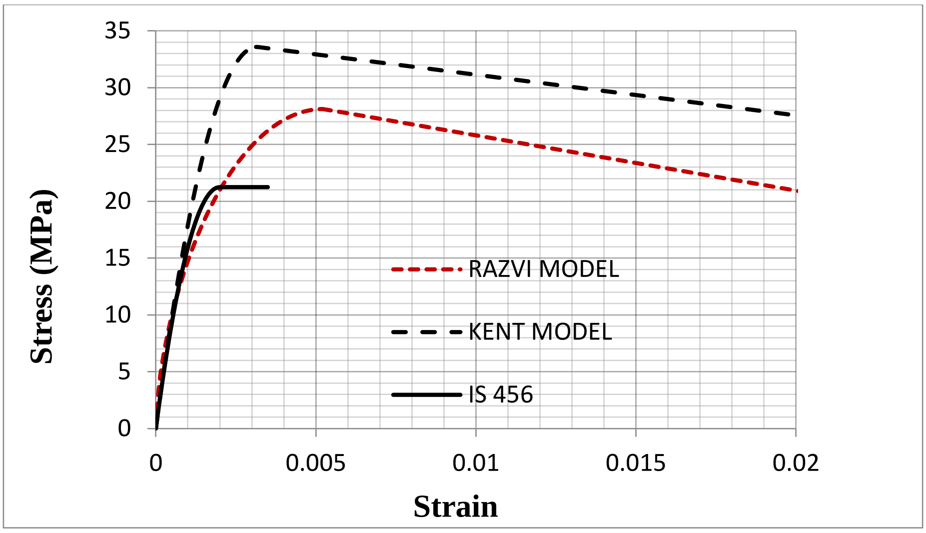


Fig: 3.11: Comparison of Stress Strain Curves Of Confined Concrete of 450C-4S4B-SM ($K = 1.58$, $K_1 = 6.67$) Section between Razvi Model, Kent Model and IS 456

The details of the stress strain values for the unconfined and confined RC sections of the frames studied using Modified Kent and Park model is shown in Table 3.7 and Table 3.8.

The experimental study conducted by Hoshikuma *et al.* (1996) suggested that as the compressive stress falls below 50% of peak strength, the core concrete crushes and the buckling of longitudinal reinforcement occurs. Since this damage is not repairable and beyond limit, it is reasonable to assume the ultimate strain as the strain corresponding to 50% of peak strength.

Table 3.7: Stress Strain Values of Unconfined Column Sections as per Modified Kent and Park Model

Peak		**50 % of Peak**		**Ultimate**	
Stress (Mpa)	**Strain**	**Stress (Mpa)**	**Strain**	**Stress (Mpa)**	**Strain**
21.25	0.002	10.625	0.0044	4.25	0.00585

Table 3.8: Stress Strain Values of Confined Column Sections as per Modified Kent and Park Model

Section Tag	K	Peak		50% of Peak		Ultimate	
		Stress (MPa)	Strain	Stress (MPa)	Strain	Stress (MPa)	Strain
400C 2S4B SM	1.4654	31.1407	0.002931	15.57	0.0365	6.228	0.059
450C 2S4B OM	1.0940	23.25	0.00218	11.62	0.008	4.651	0.013
450C 4S4B SM	1.5803	33.58	0.00316	16.796	0.0492	6.72	0.0783
500C 4S4B OM	1.1002	23.38	0.0022	11.69	0.0101	4.6761	0.01476
550C 8S4B SM	1.5141	32.175	0.00302	16.087	0.052	6.435	0.08321
650 C 8S4B 0M	1.0730	22.812	0.00214	11.406	0.0094	4.5624	0.0135
600 C 12S4B SM	1.3206	28.06	0.002641	14.031	0.0365	5.6127	0.05681
700 C 12S4B OM	1.0670	22.684	0.0021	11.34	0.0091	4.5368	0.0132

3.4.3 Limiting Values of Stress and Strain

Taking into account the spalling of the concrete cover if in case the strain outside the confined core exceeds the ultimate compressive strain of unconfined concrete, Priestley (1997) suggested an ultimate concrete strain of unconfined concrete, ε_{cu}= 0.005. This limiting value is adopted in present study. The ultimate compressive strain of confined concrete as defined in ATC-40 is given below.

$$\varepsilon_{cu} = 0.005 + \frac{0.1\rho_s f_{yh}}{f'_{co}} \leq 0.02 \qquad (3.28)$$

From the research conducted by Mondal *et al.* (2012), it was suggested that in-order to avoid the buckling of longitudinal reinforcement bars in between two successive transverse reinforcement hoops, ultimate compressive strain of confined concrete can be restricted to the limiting value of 0.02 as per the ATC-40 specifications. Thus in the present study an ultimate concrete strain of unconfined concrete, ε_{cu}= 0.005 and ultimate compressive strain of confined concrete, ε_{cc} = 0.02 is adopted.

3.5 SUMMARY AND CONCLUSIONS

First part of this Chapter deals with various confinement models for the stress-strain relationship of concrete. The confinement in the concrete plays a major role in the strength and ductility of the RC members. In order to show the effect of considering the confinement in the stress-strain curve and its effects in the strength and ductility, various sections specially detailed for confinement has to be designed. Hence a number of building frames are considered and designed as both Special Moment Resisting Frames (SMRF) and Ordinary Moment Resisting Frames (OMRF). The configuration of the frames and the reinforcement details of RC sections are also presented in this Chapter. Confinement stress-strain curves for various SMRF and OMRF sections are also developed as per various available models.

A review of various confinement models used for the stress-strain relation of concrete is also done later in this Chapter. The details of the building configuration, reinforcement details and the nomenclature assigned are shown in tabular form.

The various existing stress-strain models are studied in-order to evaluate their relative differences in representing the actual strength and deformation behaviour of confined concrete. It has been noted that the stress-strain model suggested by IS 456 does not consider the strength enhancement due to confinement while in reality concrete exhibits different performance in the confined and unconfined conditions. The model proposed by Mander *et al* (1988a) included the strength enhancement factor achieved through confinement, but it does not control the descending branch of the stress strain curve well. While comparing Razvi model (1992) and Modified Kent and Park model (1982) it was observed that the latter shows higher percentage increase in column capacity and deformation.

It was found that many research conducted show that the Modified Kent and Park model is close to the experimental results. In the present study Modified Kent and Park model (1982) has been used. Percentage Strength enhancement due to confinement in Modified Kent and Park model for various column sections is in the range of 32% – 58%. ATC-40 suggested a limiting value of ultimate strain for confined concrete as 0.02. The limiting value of ultimate strain for unconfined concrete is 0.005 as suggested by Priestly (1997).

The parametric study on Modified Kent and Park model showed that the ultimate strain is more dependent on the spacing of transverse reinforcement than the grade of transverse steel and concrete. Hence to ensure the ductile detailing, the spacing of stirrups shall be treated as an important factor.

The increase in strength enhancement factor (that define the measure of confinement) by 1.2 times increases the ultimate strain by 46.89%.

CHAPTER 4

RESPONSE REDUCTION FACTORS FOR SMRF AND OMRF FRAMES

4.1 GENERAL

The second objective of the present study is to evaluate the response reduction factors for buildings designed and detailed as per IS code. The elastic forces are reduced by a response reduction factor to calculate the seismic design base shear. The building shall be detailed as special moment resisting frames (SMRF) if the *R* factor assumed is 5. Once the design is being done, it is required to ensure that the designed building exhibit the adequate behaviour factors or response reduction factors. The actual response reduction factors can be calculated using a pushover analysis, modelling the nonlinearity in the materials. This chapter discusses the nonlinear modelling, static push over analysis of the designed RC frames (SMRF and OMRF) and the estimation of response reduction factors.

4.2 RESPONSE REDUCTION FACTOR

Chugh (2004) conducted ductility studies on RC beams using several confinement models.The response of a statically determinate structure to stress will be linear until yielding takes place. But as soon as the yielding occurs at any section, the behaviour of the structure becomes inelastic and linear elastic structural analysis can no longer be applied. As per the above study, it is mentioned that during an earthquake, yielding of the reinforcement can be expected at many sections. It would be too costly to design a structure based on the elastic spectrum. To reduce the seismic loads, IS 1893 introduces a "response reduction factor" *R*. But this reduction can be made, only if adequate ductility is developed through proper design and ductile detailing of the elements. So in-order to obtain the exact response, it is recommended to perform Non-Linear Analysis.

4.3 MODELLING OF RC MEMBERS FOR NONLINEAR STATIC ANALYSIS

OpenSees (Open System for Earthquake Engineering Simulation) platform is used for modelling of the structure.OpenSees is an object oriented open-source software framework used to model structural and geotechnical systems and simulate their earthquake response. It is primarily written in C++ and uses some FORTRAN and C numerical libraries for linear equation solving, and material and element customs. The progressive capabilities for modelling and analysing the nonlinear response of systems using a wide range of material models, elements, and solution algorithms makes this open source platform more popular.

Concrete behaviour is modelled by a uniaxial modified Kent and Park model with degrading, linear, unloading/reloading stiffness no tensile strength. Steel behaviour is represented by a uniaxial Giuffre‑Menegotto‑Pinto model. The strain hardening ratio is assumed as 5%. Fiber Section modelling of element is done according to Spacone *et.* al, (1996).The ultimate strain for confined concrete is taken as 0.02 as per ATC-40 specifications and that for unconfined concrete is considered as 0.005 as per Priestley (1997).

4.4 PUSHOVER ANALYSIS

Pushover analysis is a static, nonlinear procedure to analyse the seismic performance of a building where the computer model of the structure is laterally pushed until a specified displacement is attained or a collapse mechanism has occurred as shown in Fig: 4.1.The loading is increased in increments with a specific predefined pattern such as uniform or inverted triangular pattern. The gravity load is kept as a constant during the analysis. The structure is pushed until sufficient hinges are formed such that a curve of base shear versus corresponding roof displacement can be developed and this curve known as pushover curve. A typical Pushover curve is shown in Fig 4.1. The maximum base shear the structure can resist and its corresponding lateral drift can be found out from the Pushover curve.

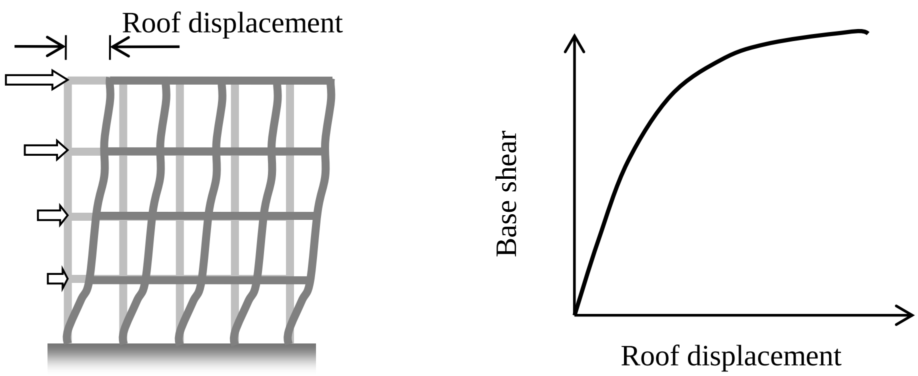


Fig: 4.1: Lateral Load Distribution and a Typical Pushover Curve

4.4.1 Bilinear Approximation of Pushover Curve

Most pushover methods adopt a bilinear approximation of the actual push-over curve to obtain an idealized linear response curve, as shown in Fig: 4.2. This is done in such a way that the area under the actual curve will be equal to the area under the bilinear approximate curve.

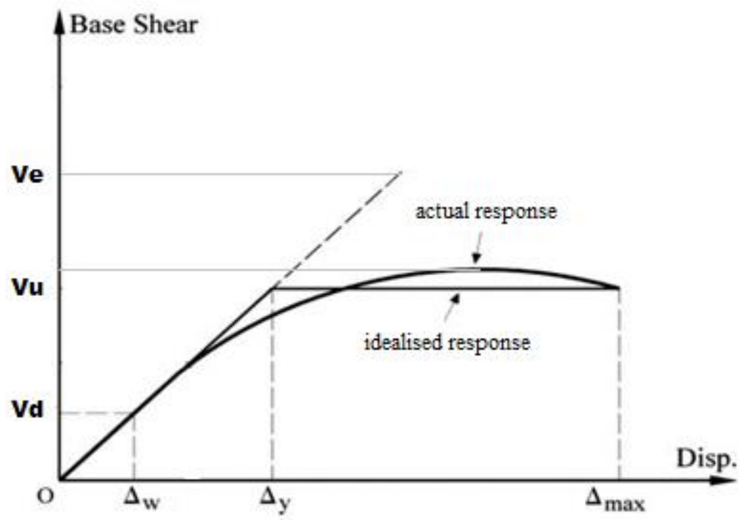


Fig: 4.2: Bilinear Approximation of Pushover Curve

4.4.2 Pushover Curves

Pushover analysis is conducted for all the frames considered in the study in-order to evaluate their seismic performance in terms of ductility capacity and over-strength. The computational model of the structure was created using the software framework, OpenSees. The gravity loads are applied as load controlled procedure and subsequently a lateral pushover analysis is conducted using a displacement controlled procedure until the maximum compressive strain in any of the members reach a value suggested by ATC-40 (Equation 3.8).

4.4.3 Effect of confinement model for concrete in lateral load behaviour

It can be seen from the previous Chapter that the effect of confinement significantly change the peak strength and ultimate strain of the stress-strain curve of concrete. In order to study the effect of concrete confinement in the pushover curve, pushover analysis of the 12 storeyed SMRF frame is conducted by modelling the concrete in the confined core using the two concrete stress-strain models namely, modified Kent and Park model and also the unconfined stress-strain model suggested by IS 456 (2000). Fig. 4.3 shows the pushover curves for the selected frame in both cases. It can be seen that difference in strength between the two pushover curves is only marginal but the change in the displacement capacity is significant. The pushover curve that uses the unconfined stress-strain model underestimates the displacement capacity of 12 storey SMRF frames by 83%. As the accuracy of displacement capacity estimation plays a major role in the estimation of response reduction factors, the SMRF and OMRF frames are modelled by the confinement model and subsequent sections explains the further details.

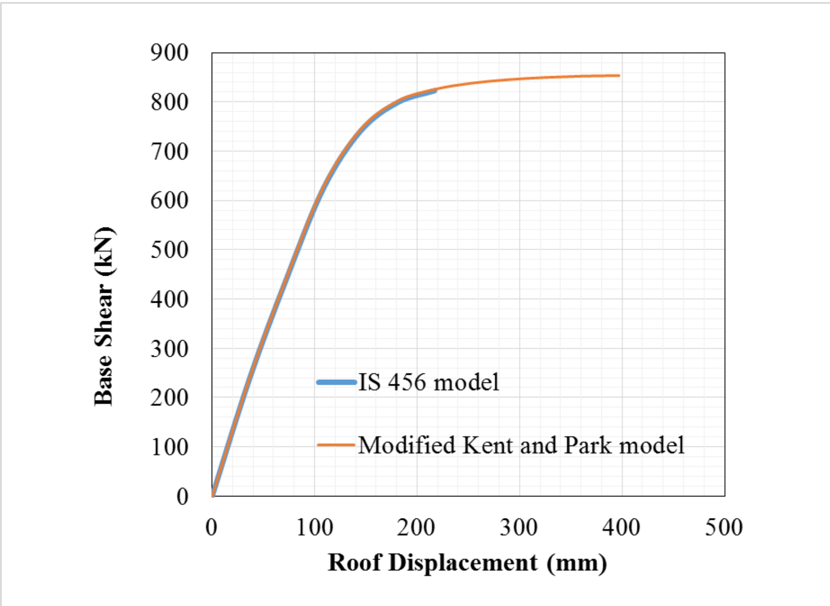


Fig: 4.3: Effect of confinement in lateral load behaviour of 12 storeyed SMRF frames

4.4.4 Comparison of Pushover curves for SMRF and OMRF buildings

The pushover curves obtained for the eight study frames are shown in Fig: 4.4(a), 4.4(b), 4.4(c), and 4.4(d). Fig. 4.4(a) shows the comparison of pushover curve for OMRF and SMRF frames for 2S4B frame. The strength capacity of OMRF frame is about 33.88 % more than that of an SMRF frame. The displacement capacity for the two storey frame detailed as SMRF frame is about 47.44% higher than that of the OMRF frame. The difference in the strength capacity is due to the increase in longitudinal reinforcement of the OMRF frame compared to that of SMRF frame. The SMRF is designed for a lower design base shear as the response reduction factor assumed is 5 instead of 3 for the OMRF frame. The same trend is followed other frames also as seen in the Figs. 4.4(b), 4.4(c), and 4.4(d). OMRF structures possess 10-34% more capacity than SMRF in resisting base shear. This is because of the fact that OMRF frames are designed with R factor '3' and the amount of longitudinal reinforcement is higher compared to SMRF. It can also be noted from the curves that the maximum displacement shown by SMRF frames is higher in all the cases compared to their corresponding OMRF frames as a result of the enhanced confinement achieved through special design and ductile detailing. SMRF buildings exhibit about 30-65% more deformation capacity than OMRF buildings.

Table 4.1 summarizes the percentage increase in roof displacement capacity and base shear of both SMRF and OMRF frames. In order to show the trend of increase in strength and displacement capacity of OMRF/SMRF compared to each other a trend-line graph is plotted in Fig: 4.5 and Fig:4.6. The trend-line show that (Fig: 4.5), as the number of storeys increases the strength increase of OMRF compared to SMRF decreases. Similarly, as the displacement capacity increase in SMRF frame compared to an OMRF frame decreases as the number of storeys increases (Fig: 4.6).

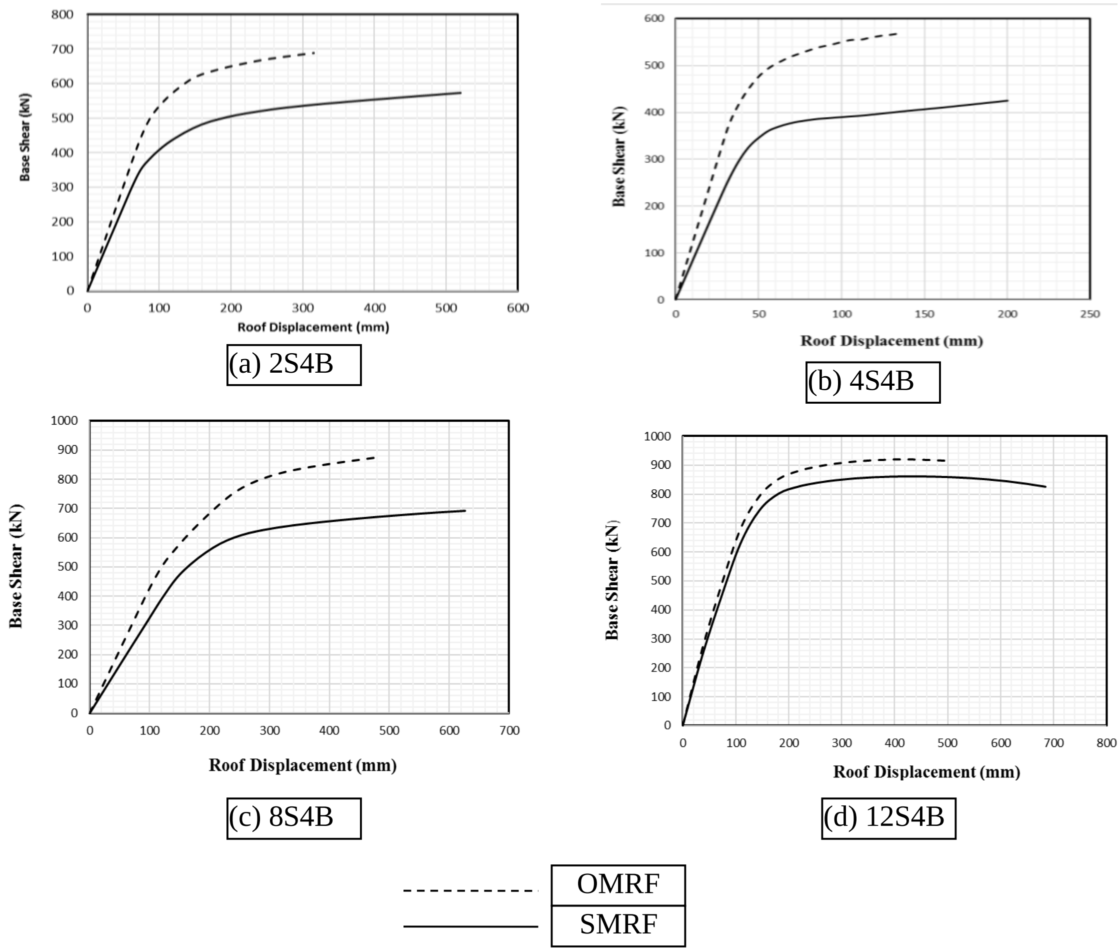
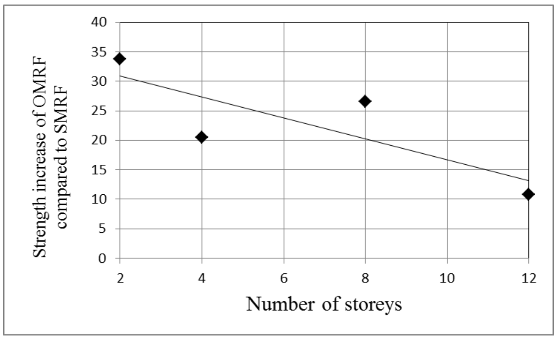


Fig: 4.4: Pushover curve of SMRF and OMRF frames

Table 4.1: Comparison of strength and deformation capacity for SMRF and OMRF frames

No. of storeys	Roof Displacement (mm)		Percentage Increase in Roof Displacement Capacity of SMRF	Base Shear (kN)		Percentage Increase in Base Shear of OMRF
	OMRF	SMRF		OMRF	SMRF	
2	135.82	200.23	47.44 %	569.41	425.5	33.88 %
4	316.02	520.284	64.64 %	689.04	572.4	20.45 %
8	483.369	626.36	29.59 %	876.02	692.8	26.58 %
12	505.212	684.76	35.64 %	914.47	825.87	10.736 %



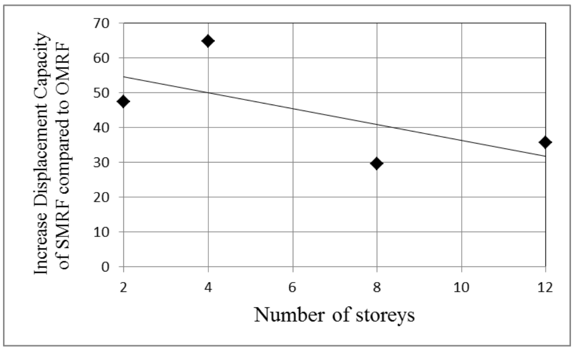


Fig: 4.5: Strength increase of OMRF compared to SMRF

Fig: 4.6: Displacement increase of SMRF compared to OMRF

4.4.5Effect of number of stories and frame type on seismic performance

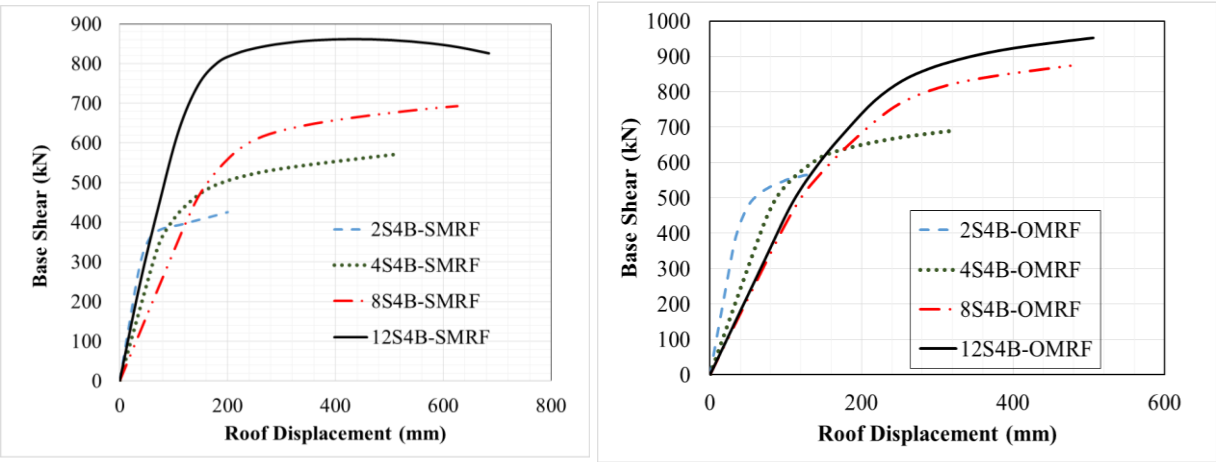
In order to study the effect of number of storeys in the strength and displacement capacity of frames considered, their respective pushover curves are being compared. Fig. 4.6a and 4.6b display the pushover curves of SMRF and OMRF frames respectively.

SMRF frames

With respect to SMRF frames, the strength of the 12 storeyed frames is about 24.9% more than that of 8 storeyed frame, which is about 20.55% more than that of 4 storeyed frame, which is about 34.5% more than that of 2 storied frames. When the number of storeys is increased by 4 storeys the base shear capacity is increased by 20-25%. The displacement capacity of the 12 storeyed frames is about 13.7% more than that of 8 storeyed frame, which is about 15.8% more than that of 4 storeyed frame, which is about 16% more than that of 2 storied frames. For an SMRF frame, when the number of stories increased by 4 storeys the displacement capacity is increased by 13-15%.

OMRF frames

With respect to OMRF frames, the strength of the 12 storeyed frames is about 8.7% more than that of 8 storeyed frame, which is about 27.1% more than that of 4 storeyed frame, which is about 23% more than that of 2 storied frames. The displacement capacity of the 12 storeyed frames is about 4.6% more than that of 8 storeyed frame, which is about 53% more than that of 4 storeyed frame, which is about 185% more than that of 2 storied frames. For an OMRF frame, when the number of stories increased by 4 storeys the displacement capacity is increased by 4-53%.



(a) SMRF

(b) OMRF

Fig: 4.7: Effect of number of storeys on the pushover curves

4.5 RESPONSE REDUCTION FACTOR as per IS 1893 (2002)

As per the IS 1893 definition, "it is the factor by which the actual base shear force, that would be generated if the structure were to remain elastic during its response to the Design Basis earthquake (DBE) shaking, shall be reduced to obtain the design lateral force".

When a structure is subjected to seismic loads, a base shear which is prominently higher than the actual structure response is created. Thus it possess a significant amount of reserve strength or over-strength. Over-strength is developed because the maximum lateral strength of a structure always exceeds its design strength. Once it enters the inelastic phase, it is capable of resisting and absorbing a large amount of seismic energy. Hence seismic codes introduce a reduction in the design loads, taking benefit of the fact that the structure possesses over-strength and ductility as per Asgarian and Shokrgozar (2009).This force reduction factor is called Response Reduction Factor, R.

4.5.1 Behaviour factors (Performance parameters)

The response modification factor or response reduction factor is a measure of the over strength and ductility of the structure in inelastic phase. This is also called as behaviour factor in Inter-national codes, and it can be expressed as a function of various parameters of the structural system, such as strength, ductility, damping and redundancy as per Whittaker *et al.* (1999).

$$R = R_s R_\mu R_R R_\xi \tag{4.1}$$

Where R_s is the strength factor, R_μ is the ductility factor, R_ξ is the damping factor, and R_R is the redundancy factor. The over-strength factor R_s is a measure of the reserve strength in the structure. It is defined as the ratio of maximum base shear in the actual non-linear behaviour V_u to the design base shear V_d. The ductility factor R_μ is a measure of the deformation capacity of the structure. It is obtained as the ratio of the elastic base shear (V_e) to the ultimate base shear of the inelastic response (V_u). The damping factor, R_ξ balances the effect of supplementary viscous damping and is mainly applicable in the case of structures with additional energy dissipating devices. In the absence of such devices the damping factor is generally assumed as 1.0. From the studies conducted by Mondal *et al.* (2013) redundancy factor R_R can be assumed as unity following the ASCE7 guidelines. The ductility capacity $\boldsymbol{\mu}$ is defined as the ratio of the maximum deformation to the displacement corresponding to yield strength of the idealized elastic response. The governing equations for the estimation of behaviour factors used in the current study are given below.

$$R = \frac{\mathrm{Ve}}{\mathrm{Vd}} = \frac{\mathrm{Vu}}{\mathrm{Vd}} \times \frac{\mathrm{Ve}}{\mathrm{Vu}} \tag{4.2}$$

$$= R_s R_\mu \tag{4.3}$$

$$\mu = \Delta_u / \Delta_y \tag{4.4}$$

The details of the behaviour factors are calculated for the SMRF buildings as shown in Table 4.2 and Table 4.3.

Table 4.2: Parameters of the pushover curves for SMRF and OMRF Frames

Frame	$\boldsymbol{\Delta_u}$(mm)	$\boldsymbol{\Delta_y}$(mm)	$\boldsymbol{V_u}$(kN)	$\boldsymbol{V_d}$(kN)	$\boldsymbol{\mu = \frac{\Delta_u}{\Delta_y}}$	$\boldsymbol{\Omega = \frac{V_u}{V_d}}$
SMRF Frames						
2S4B	200.23	50.02	425.52	212.02	4.00	2.01
4S4B	520.28	110.02	572.41	321.36	4.73	1.78
8S4B	626.36	200.12	692.8	431.6	3.13	1.61
12S4B	612.93	155.64	861.64	505.87	3.94	1.70
OMRF Frames						
2S4B	135.02	43	569.41	380.47	3.139	1.49
4S4B	316.02	106	689	540.8	2.981	1.27
8S4B	483.369	180	876.029	745.26	2.55	1.36
12S4B	505	190	952.5	853.03	2.65	1.116

Table 4.3: Response reduction factors and the components (Behaviour factors)

Frame	$\boldsymbol{R_s}$	$\boldsymbol{R_\mu}$	$\boldsymbol{R_R}$	$\boldsymbol{R}$
SMRF frames				
2S4B	2.007	2.42	1	4.856
4S4B	1.781	2.71	1	4.827
8S4B	1.605	2.63	1	4.229
12S4B	1.703	2.52	1	4.305
OMRF frames				
2S4B	1.49	2.007	1	2.99
4S4B	1.27	2.062	1	2.63
8S4B	1.176	1.893	1	2.226
12S4B	1.116	1.974	1	2.202

4.5.2 Performance parameters versus number of storeys (SMRF and OMRF frames)

Variation of over-strength factors for SMRF and OMRF frames with number of storeys

In order to understand the variation of each performance parameter with number of storeys of each SMRF and OMRF frame, the data in the presented in the Table 4.3 is expressed graphically. With reference to Figs: 4.7(a) & 4.7(b), as the number of storeys increases the over-strength factor, R_S show a decreasing trend for both SMRF and OMRF frames. Thus the shorter frames possess higher over-strength factor compared to the taller frames. This type of behaviour is also observed by Mondal *et al*. (2013) and Asgarian and Shokrgozar (2009).

Variation of ductility factor for SMRF and OMRF frames with number of storeys

Figs: 4.7(c) and 4.8(d) show the variation in ductility, R_μ with the number of storeys for both the frames. It looks like there is no definite trend for ductility factor as the number of stories increases in both SMRF and OMRF frames.

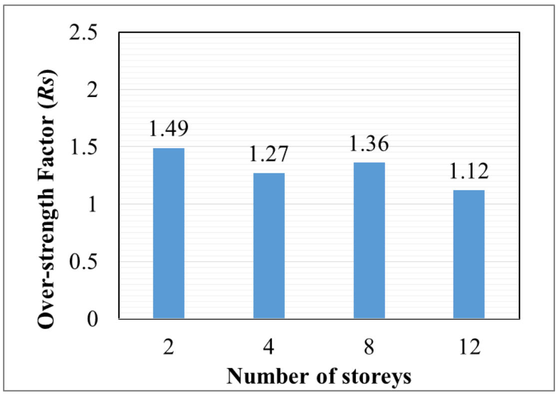
Variation of response reduction factor for SMRF with number of storeys

Fig: 4.7(e) and 4.7(f) show the variation in Response Reduction Factor, *R* with the number of storeys. It can be seen that as the number of storeys increases the response reduction factor decreases for both SMRF and OMRF frames.
Two storey SMRF frame shows the highest *R* factor of 4.86while eight storey SMRF frame shows the lowest value of 4.23, the values being close to the design *R* factor of 5. The *R* values vary within the range 4.23 to 4.86 for the SMRF frames considered which is 2.8 to 15.6 % less than the assumed value of *R* during the design.

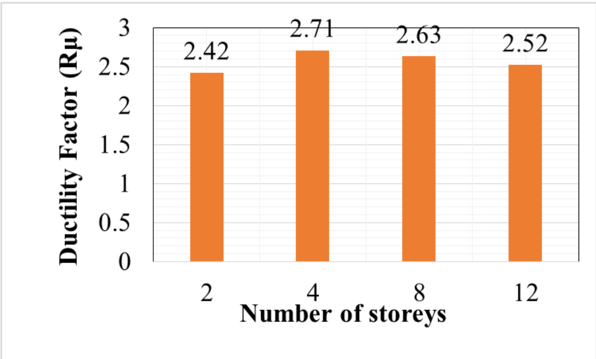
Variation of response reduction factor for OMRF with number of storeys

Two storey OMRF frame shows the highest *R* factor of 2.99while twelve storey OMRF frame shows the lowest value of 2.2, the values being slightly less than the design *R* factor of 3 for OMRF frames. The *R* values vary within the range of 2.2 to 2.99 for the OMRF frames considered which is 0.33% – 26 % less than the assumed value of *R* during the design.

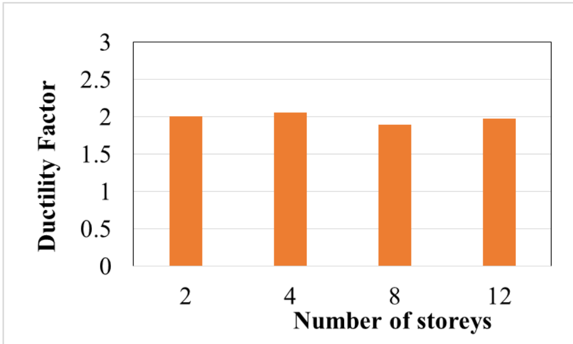
(a) Over-strength factor -SMRF



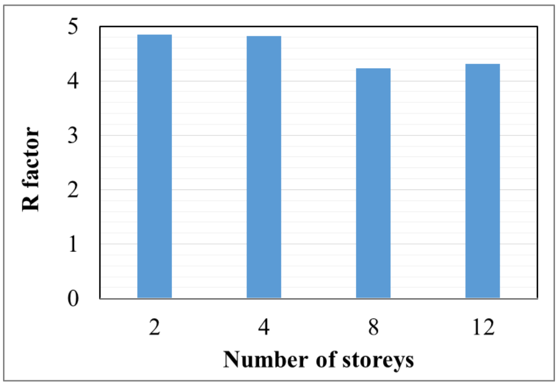
(b) Over-strength factor -OMRF



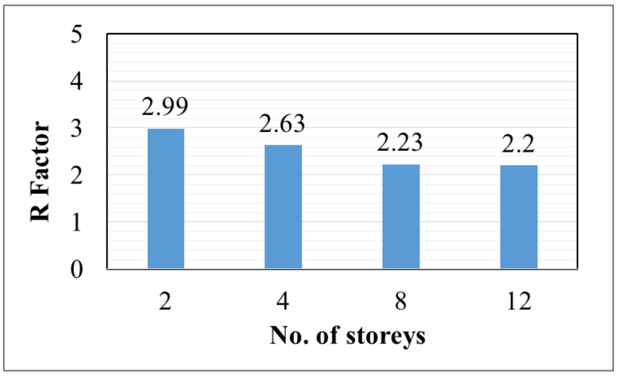
(c) Ductility factor -SMRF



(c) Ductility factor -OMRF



(e) Response reduction factor -SMRF



(f) Response reduction factor -OMRF

Fig: 4.8: Variation of Performance parameters for SMRF and OMRF frames with number of stories

4.6 CONCLUDING REMARKS

The objective of this Chapter is to estimate the response reduction factors for the specially and ordinary moment resisting frames. All the RC frames are modelled for nonlinearity using the Modified Kent and Park confinement model. Nonlinear Static Pushover Analysis is carried out for all the frames considered to evaluate the performance factors.

The pushover analysis of the 12 storeyed SMRF frame modelling the concrete in the confined core using the two concrete stress-strain models namely, modified Kent and Park model shows that the unconfined stress-strain model underestimates the displacement capacity of 12 storey SMRF frames by 83%.

The pushover curves of SMRF buildings are compared with that of their corresponding OMRF buildings. It is observed that the drift capacity of SMRF buildings is higher than OMRF buildings in all the cases. The percentage increase of displacement capacity of SMRF over OMRF varies in the range 29-65%. This validates the fact that SMRF buildings which are specially designed and detailed as per IS 13920 guidelines exhibits more ductility compared to the less stringently designed OMRF buildings. While considering the base shear capacity, OMRF buildings exhibit higher values than SMRF buildings of about 10-34%.The provision of *R* factor '3' increases the design base shear in OMRF buildings. Due to the higher design base shear, the RC sections in the OMRF building will be heavier. This is the reason for the higher base shear capacity.

The behaviour factors of the frames are evaluated from the pushover curve and a story-wise comparison is carried out. For both SMRF and OMRF buildings it is found that the over-strength factors exhibits a decreasing trend as the number of stories increases. The shorter frames show higher over-strength value compared to taller frames.

It was found that the ductility factors do not show any specific trend with variation in the number of stories for both SMRF and OMRF frames.

A study of the variation of response reduction factor with number of stories is done. In SMRF buildings it is observed that as the number of storeys increases the *R* factor tends to decrease. The shorter frames exhibits higher *R* values compared to taller frame. 2-storey. SMRF building shows the highest *R* factor of 4.856 which is almost close to the IS 1893(2002) suggested value of '5'. The R factor for SMRF buildings varies in the range of 4.23 to 4.86. OMRF buildings also exhibit decrease in *R* factor with increase in number of storeys. The value varies in the range 2.2 to 2.99 which is less than the suggested *R* value of '3' as per IS 1893 guidelines.

In general, the present study shows that both the OMRF and SMRF frames failed to achieve the respective target values of response reduction factors recommended by IS 1893 (2002). Further research is required in this direction by considering more spectrum of frames designed as per the two approaches (SMRF and OMRF) in IS code, before reaching any specific conclusions about the adequacy of the codal requirements.

The effect of number of storeys in the base shear strength and displacement capacity of the SMRF and OMRF frames is studied. It is found that for addition of every 4 storeys in the SMRF frames, it showed about 20-25% increase in base shear capacity while about 13-15% increase in displacement capacity.

CHAPTER 5

SUMMARY AND CONCLUSIONS

5.1 REVIEW OF EXISTING CONFINEMENT MODELS FOR CONCRETE

Objectives of the thesis are to review the existing confinement models for concrete and to apply an appropriate confinement model to SMRF and OMRF buildings designed as per IS 1893 (2002). A literature is conducted that discusses the various topics such as the confinement models, response reduction factors or behaviour factors and various confinement models for the stress-strain relationship of concrete and pushover analysis.

The confinement in the concrete plays a major role in the strength and ductility of the RC members. In order to show the effect of considering the confinement in the stress-strain curve and its effects in the strength and ductility, various SMRF and OMRF frames (2, 4, 8 and 12 storeys with 4 bays) are designed and detailed as per IS code.

The various existing stress-strain models are studied in-order to evaluate their relative differences in representing the actual strength and deformation behaviour of confined concrete. It has been noted that the stress-strain model suggested by IS 456 does not consider the strength enhancement due to confinement while in reality concrete exhibits different performance in the confined and unconfined conditions.

A parametric study is conducted to understand how the various parameters such as spacing transverse reinforcement, grade of transverse reinforcement and grade of concrete influence the stress-strain curve.

- It was found that Razvi model and Modified Kent and Park model it was observed that the latter shows higher percentage increase in column capacity and deformation. Percentage Strength enhancement due to confinement in Modified Kent and Park model for various column sections is in the range of 32% –58%.
- The parametric study on Modified Kent and Park model showed that the ultimate strain is more dependent on the spacing of transverse reinforcement than the grade of transverse steel and concrete. Hence to ensure the ductile detailing, the spacing of stirrups shall be treated as an important factor. The

increase in strength enhancement factor (that define the measure of confinement) by 1.2 times increases the ultimate strain by 46.89%.

5.2 PUSHOVER CURVES FOR SMRF AND OMRF FRAMES

The second objective is to estimate the response reduction factors for the specially and ordinary moment resisting frames. The designed RC frames are modelled for nonlinearity using the Modified Kent and Park confinement model. Nonlinear Static Pushover Analysis is carried out for all the frames to generate the pushover curves.

- The pushover analysis of the 12 storeyed SMRF frame modelling the concrete in the confined core using the two concrete stress-strain models namely, modified Kent and Park model shows that the unconfined stress-strain model (IS code) underestimates the displacement capacity of 12 storey SMRF frames by 83%.

- The pushover curves of SMRF buildings are compared with that of their corresponding OMRF buildings. It is observed that the drift capacity of SMRF buildings is higher than OMRF buildings in all the cases.

- The percentage increase of displacement capacity of SMRF over the corresponding OMRF is in the range of 29-65%. This validates the fact that SMRF buildings which are specially designed and detailed as per IS 13920 guidelines exhibits more ductility compared to the less stringently designed OMRF buildings.

- While considering the base shear capacity, OMRF buildings exhibit higher values than SMRF buildings of about 10-34%.The provision of R factor ‘3’ increases the design base shear in OMRF buildings. Due to the higher design base shear, the RC sections in the OMRF building will be heavier. This is the reason for the higher base shear capacity.

- The behaviour factors of the frames are evaluated from the pushover curve and a story-wise comparison is carried out. For both SMRF and OMRF buildings it is found that the over-strength factors exhibits a decreasing trend as the number of stories increases. The shorter frames show higher over-strength value compared to taller frames.

- It was found that the ductility factors do not show any specific trend with variation in the number of stories for both SMRF and OMRF frames.

5.3 RESPONSE REDUCTION FACTORS FOR SMRF AND OMRF FRAMES

A study of the variation of Response Reduction Factor with number of stories is conducted. In SMRF buildings it is observed that as the number of storeys increases the *R* factor tends to decrease. The shorter frames exhibits higher *R* values compared to taller frame. 2- storey SMRF building shows the highest *R* factor of 4.856 which is almost close to the IS(1893) code suggested value of ‘5’.

- The *R* factor for SMRF buildings varies in the range of 4.23 to 4.86. OMRF buildings also exhibit decrease in *R* factor with increase in number of storeys. The value varies in the range 2.2 to 2.99 which is less than the suggested *R* value of ‘3’ as per IS 1893 guidelines.

- In general, the present study shows that both the OMRF and SMRF frames, failed to achieve the respective target values of response reduction factors recommended by IS 1893 (2002).

- The study of effect of number of storeys in the base shear strength and displacement capacity of the SMRF and OMRF frames show that for addition of every 4 storeys in the SMRF frames, it showed about 20-25% increase in base shear capacity while about 13-15% increase in displacement capacity.

5.4 LIMITATION OF PRESENT STUDY AND SCOPE FOR FUTURE WORK

The present study considered frames with number of storeys varying from two, four, eight and twelve with four number of bays. The aspect ratios of (ratio of height to width) of each frames considered is not the same. The trend of *R* factors and the components of *R* factors show some exceptions in the decreasing rend in some cases. The selection of frames with same aspect ratio may yield variation of *R* factors with some specific trend. The present study can be extended to frames with same aspect ratios.

The present study does not consider the effect of strength and stiffness of infill walls in the frames. This approach can be extended to frames modelling the infill walls.

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