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DEFORMATION AND STRENGTH BASED ASSESSMENT OF
SEISMIC FAILURE MECHANISMS FOR EXISTING RC FRAME
BUILDINGS

A Dissertation Submitted in Partial
Fulfilment of the Requirements for the Master Degree in
EARTHQUAKE ENGINEERING

by

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The dissertation entitled “Deformation and Strength Based Assessment of Seismic Failure Mechanisms for Existing RC Frame Buildings”, by Ahmad Abo El Ezz, has been approved in partial fulfilment of the requirements for the Master Degree in Earthquake Engineering.

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ABSTRACT

The motivation for the presented research work is the need for a reliable and simplified assessment method for predicting the most probable side-sway failure mechanism for the existing RC frame buildings, which is a basic step in evaluating the seismic capacity of these buildings. In order to predict the most probable failure mechanism, the concept of the sway potential index is introduced. A strength based sway index, as well as a newly proposed deformation based index, are verified. The concept of the strength based index is to check whether plastic hinges occur in beams or columns by comparing the sum of the probable flexural strengths of the beams to the columns. The deformation based sway index is introduced considering the relative flexibility of beams and columns using only the geometrical properties as an indicator of potential failure mechanisms. The definition of the most probable side-sway mechanism of the frame is evaluated by comparing the yield rotation of the columns to those of the beams. The deformation based index is a measure of the dominant flexible members that are expected to control the behaviour of the frame. The present study has verified the suggested sway potential indices using a newly developed displacement based nonlinear adaptive pushover analysis method and suggests limiting values between beam sway and column sway mechanisms for bare as well as masonry infilled existing RC frame buildings. Additionally, storey strength and stiffness ratios are developed as additional information to help in the prediction of the failure mechanism, which reflects the vertical distribution of strength and stiffness along the height of the building.

Keywords: Seismic performance assessment, Failure mechanisms, Adaptive pushover analysis, Sway potential index, Masonry infilled frames.
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1 INTRODUCTION

1.1 Background

Protection against failure has always been a major objective of seismic design. Failure refers to the loss of ability of a structural system, or any part, to resist tributary gravity loads. Local failure may occur, for instance, if a vertical load-carrying component fails in compression, or if shear transfer is lost between horizontal and vertical components (e.g., shear failure between a flat slab and a column). Global, or at least storey failure will occur if local collapses propagate or if an individual storey displaces sufficiently so that the second-order P delta effects fully offset the first-order storey shear resistance and dynamic instability (side-sway failure) occurs (Krawinkler and Zerian 2007).

Failure prediction has become a popular research subject during the last decade. Several shaking table studies have been performed on full-scale and reduced-scale replication of multi-storey reinforced concrete and steel buildings, and analytical modelling of response until collapse has become a common practice. It should be realized that failure prediction is not a deterministic process. Ground motion and modelling uncertainties will always dominate the process. It seems difficult to predict what type of failure would occur under different types of severe earthquake ground motions. It is important to develop more rational and reliable seismic design and assessment methods to deepen our understanding of failure behaviour and the ultimate states of various types of buildings structures.

In terms of the global failure analysis, the two basic simple failure mechanisms are a column side-sway mechanism when column critical sections develop yield before beams and a beam side-sway mechanism when beams commence yield before columns. More specifically, a column sides-way mechanism is defined such that plastic hinges form at the top and bottom of all columns of a storey. A beam side-sway mechanism is defined when plastic hinges form at critical sections of all beams in the frame and the bottom of first storey columns. For most buildings, a mixed side-sway mechanism will probably be the actual failure mechanism. In this work, since the approximate failure mechanism was determined by using sway potential indices, only the column side-sway and beam side-sway mechanisms were pursued to determine how close the prediction is to the more accurate mechanism predicted by pushover analysis.
1.2 **Objective and scope**

In this research work, a strength based sway potential index suggested by Priestley (1997) as well as a newly proposed deformation based index are verified to help in definition of a rational and reliable method of predicting the most probable side-sway mechanism, which is a basic step in evaluating the seismic capacity of existing reinforced concrete buildings. The sway indices will be applied to both bare frames and masonry infilled frames in order to define limiting values between the beam sway and column sway mechanisms. Additionally, storey strength and stiffness ratios are developed to add more information to predict the side-sway mechanism, which reflect the vertical distribution of strength and stiffness along the height of the frame. The newly developed displacement based nonlinear adaptive pushover analysis method is used in the verification of the predicted mechanism using the sway indices and storey ratios.

Another objective of this work is to study the effects of the masonry infill panels in the changing the global side-sway mechanism and consequently the seismic capacity.

The newly proposed deformation based index along with the storey stiffness ratio are expected to be used reliably to predict the side-sway mechanism and they have the advantage of being calculated using only the geometry of the frame as they rely only on the expected proportionality between the strength and stiffness. The outcome of this research work can mainly be applied to existing gravity load designed frames with no or minimum seismic provisions that represent a large proportion of the building stock in most of the seismically vulnerable countries and which have a large variability in the geometrical and material properties.

1.3 **Organisation and outline**

The second chapter of this dissertation presents a literature review of the research work on the prediction of global failure mechanisms in reinforced concrete frame buildings. The chapter contains a summary of the research on vertically irregular RC buildings and the current code procedures for evaluating deficiencies in existing buildings.

The third chapter presents the case study reinforced concrete frames that have been modelled and a discussion of the used numerical software and modelling assumptions.

The fourth chapter introduces the concept of the sway potential index and the development of both the strength and deformation based sway indices.

The fifth chapter shows the results of the nonlinear adaptive pushover analysis of the case studies bare frames using the numerical modelling approach. This chapter outlines the concept of the adaptive pushover analysis and its advantages over conventional pushover and the parameter that have been used in the assessment procedure.
The sixth chapter includes the results of the nonlinear adaptive pushover analysis of the case studies masonry infilled frames. This chapter starts with a review of the current research work on the seismic performance and assessment of the masonry infilled frames.

The seventh chapter presents a discussion and comparison of the results obtained from the prediction of sway indices and the numerical pushover analysis prediction. The chapter introduces the limits of the sway indices to predict beam sway or column sway mechanisms and the additional storey strength and stiffness ratios required for a reliable prediction of the mechanism. The comparison also shows the effect of the masonry infill panels in changing the failure mechanism of the bare frames.

Finally, the eighth chapter concludes with a summary of the present work and some guidelines for the future research developments.

One appendix is added at the end of this report that shows the results of the moment curvature section analysis of the case studies that have been modelled.
2 LITERATURE REVIEW

2.1 Review of vertical irregularities in RC frame buildings

Many buildings are in some sense vertically irregular. Some are originally constructed this way, (e.g. in the case of a soft first-storey) whilst others have become so by accident, for example due to inconsistencies or even errors during the construction process, while many have been rendered irregular during their lifetime because of damage, rehabilitation or change of use (Fragiadakis et. al. 2006). Therefore, it is essential for structural engineers to obtain a better understanding of the seismic response of structures with irregular distributions of mass, stiffness or strength along their height, a need that has also been recognized by current seismic guidelines.

Irregularities are based on the three basic quantities, mass, stiffness and strength that can differ separately or jointly, for single or multiple storeys, along the building height. According to Al-Ali and Krawinkler (1998), the most common four types of vertical irregularities are: mass, stiffness, strength and combined stiffness–strength irregularities. Mass irregularities are very common and usually are due to different use of one floor compared to adjacent ones, such as car parking floors, or floors with mechanical equipment. Typical stiffness irregularities appear as soft storeys or in general when elements of the lateral-force-resisting system, are present on one storey but not on adjacent storeys. In many practical cases, strength changes occur together with stiffness, e.g. when the cross section of a member is changed both the moment of inertia and the plastic moment capacity are modified. Vertical irregularity cases of this type also appear when a structure has a setback, or when the number or size of structural members is reduced, for example when columns are stopped at the second floor instead of terminating at the basement. Stiffness and strength can also be modified unintentionally by non-structural elements, such as partition walls.
Recently, most of the studies aim at a better understanding of the seismic response of irregular buildings in elevation in order to obtain information useful to assessing suitability of the code criteria that distinguish between vertically regular and irregular structures and special rules provided for the latter ones.

### 2.2 Review of code procedures for evaluating deficiencies in existing buildings

A comprehensive review of seismic evaluation documents published at the National Information Centre of Earthquake Engineering in India at the Indian Institute of Technology-Kanpur (Rai 2005) emphasised that all seismic evaluation procedures follow similar assessment steps which can be broadly grouped into two categories: (a) configuration-related and (b) strength related assessment. The first category involves a quick assessment of the earthquake resistance of the building and its potential deficiencies, with the objective to screen out the significantly vulnerable structures for the second detailed analysis and evaluation. The first category evaluation typically consists of assessing the configurationally induced deficiencies known for unsatisfactory performance, whereas the next level of evaluation consists of proper force and displacement analysis to assess structural performance at both global and component level. The checks related to identify potential deficiencies which may cause an undesirable failure modes are summarized in the following section.
2.2.1 Configuration related assessment

Good details and construction quality are of secondary value if a building has an irregular shape that was not properly considered in the design (Rai 2005). Although a building with an irregular configuration may be designed to meet all code requirements, irregular buildings generally do not perform as well as regular buildings in an earthquake. Typical building configuration deficiencies include an irregular geometry, a weakness in a given storey, a concentration of mass, or a discontinuity in the lateral force resisting system. Vertical irregularities are defined in terms of strength, stiffness, geometry and mass. Horizontal irregularities involve the horizontal distribution of lateral forces to the resisting frames or shear walls.

2.2.1.1 Weak Storey

The storey strength is the total strength of all the lateral force-resisting elements in a given storey for the direction under consideration. Weak storeys are usually found where vertical discontinuities exist, or where member size or reinforcement has been reduced. The result of a weak storey is a concentration of inelastic activity that may result in the partial or total collapse of the storey. According to FEMA (2000) and the New Zealand draft code (NZSEE, 2002), the strength of lateral force resisting system in any storey shall not be less than 80% of the strength in an adjacent storey, above or below.

2.2.1.2 Soft Storey

The soft storey condition commonly occurs in buildings with open fronts at ground floor or with particularly tall first storeys. Soft storeys usually are revealed by an abrupt change in interstorey drift. Although a comparison of the stiffnesses in adjacent storeys is the direct approach, a simple first step might be to compare the interstorey drifts. According to FEMA (2000) and the New Zealand draft code (2002), the stiffness of lateral force resisting system in any storey shall not be less than 70% of the stiffness in an adjacent storey above or below, or less than 80% of the average stiffness of the three storeys above or below. Eurocode 8 (CEN, 2003) specifies that there should not be significant difference in the lateral stiffness of individual storeys and at any storey, the maximum displacement in the direction of the seismic forces should not exceed the average storey displacement by more than 20%.

2.2.1.3 Geometry

Geometric irregularities are usually detected in an examination of the storey-to-storey variation in the dimensions of the lateral-force-resisting system. FEMA 310 (2000) give a quantitative check for the geometry of the building, according to which there shall be no change in the horizontal dimension of lateral force resisting system of more than 30% in a storey relative to adjacent storeys. As per New Zealand Draft Code (NZSEE, 2002), plan irregularities include irregular mass distribution, re-entrant corners and buildings with ‘wings’ that form an ‘L’, ‘T’ or ‘E’ shape. As per Eurocode 8 (CEN, 2003), the building structure should be approximately symmetrical in plan with respect to two orthogonal directions, in
what concerns lateral stiffness and mass distribution. Also if setbacks are present, the
specified provisions should be applied.

2.2.1.4 Effective Mass

Mass irregularities can be detected by comparison of the storey weights. The effective mass
consists of the dead load of the structure tributary to each level, plus the actual weights of
partitions and permanent equipment at each floor. Mass irregularities affect the dynamic
response of the structure, and may lead to unexpected higher mode effects and concentrations
of demand. Effective mass between adjacent storeys shall not vary by more than 50% as per
FEMA 310 (2000). According to the New Zealand draft code (NZSEE, 2002), a significant
vertical irregularity results when the mass of a storey varies by 30% from those adjacent.
Eurocode 8 (CEN, 2003) specifies that the individual storey mass should remain constant or
reduce gradually without abrupt changes.

2.3 Strength and stiffness considerations in seismic response

2.3.1 Effect of beam-to-column stiffness ratio on lateral displacement response

The beam-to-column stiffness ratio ($\rho$) controls the relative joint rotation in building systems
due to the beam and column flexural stiffness contributions at the storey levels. It is the ratio
of sum of the beam rigidities (EI/L)$_b$ to column rigidities (EI/L)$_c$ at the storey that is closest
to the mid-height of the building. It provides useful information for the global dynamic
structural characteristics similar to the essential knowledge prevailed by the fundamental
mode and damping. Thus, it can be a versatile parameter for modeling complex structural
systems to capture the prominent dynamic features of the entire system. The general form of $\rho$
is given by

$$\rho = \frac{\sum \text{beams}(EI/L)_b}{\sum \text{Columns}(EI/L)_c}$$  (2.1)

According to Akkar et al. (2005), the curves in Figure (2.3) show the fundamental mode
shapes and corresponding interstorey drifts of different $\rho$ values for regular frames with
uniform lateral stiffness along the height. The mode shapes are normalized with respect to the
top modal displacement. The modal interstorey drifts are normalized with respect to their
maximum values. As illustrated in Figure (2.2), the mode shape for $\rho = 0.0$ represents flexural
behaviour. As $\rho$ increases, the behaviour is controlled by both shear and flexural
displacements. When $\rho = \infty$, the structure acts as a shear frame. The normalized mode shapes
indicate that the increase in $\rho$ increases the lateral displacement for the fundamental mode.
The largest difference between the first-mode lateral displacements and interstorey drifts
occurs when $\rho$ is between 0.0 and 0.125 where the transition from flexural to combined mode
behaviour. The maximum interstorey drift shifts rapidly from the upper half of the frame to
the lower half for $\rho$ between 0.0 and 0.125. In extreme cases, when the first mode dominant
structure behaves as a flexural cantilever $\rho=0.0$ or in pure shear $\rho=\infty$, the maximum interstorey drift occurs at the top or at the ground storey, respectively. The comparison of the fundamental mode lateral displacements and interstorey drift curves for $\rho \geq 0.125$ indicates that the frame interstorey drift variation is more sensitive to the changes in $\rho$ with respect to lateral displacements.

Figure 2.2: Overall lateral deformation in buildings (Miranda 2002).

It should be noted how the maximum interstorey drift locations change along the elevation for different beam-to-column stiffness ratios. The variation of maximum interstorey drift is described in Figure (2.4) by plotting the ratio of first-mode maximum modal interstorey drift ($\Phi_{IDR_{max}}$) to modal ground storey drift ($\Phi_1$) in terms of fundamental period and different $\rho$ values as developed by Akkar et al. (2005). The ratios show the change in maximum interstorey drift with respect to ground storey drift for a given $\rho$ value. The amplification of maximum interstorey drift relative to ground storey drift increases greatly for decreasing $\rho$ values. The increase is more stable as the fundamental period becomes larger. Inherent from the theoretical first-mode behaviour, the maximum interstorey drift always occurs at ground storey level for shear frames $\rho=\infty$. 
2.3.2 Effect of column-to-beam strength ratio in capacity design concept

According to a review report of the seismic behaviour of beam-column joints which has been carried out by Uma and Jain (2005), the codes recommend expressions to prevent formation of plastic hinges in columns which essentially aim at providing stronger columns with capacity more than the flexural strength of beams obtained considering over strength factors. A rigorous interpretation of expressions requires calculation of the moments at the centre of the joint. These moments correspond to development of the design values of the moments of resistance of the columns or beams at the outside faces of the joint, plus a suitable allowance for moments due to shears at the joint faces. However, the loss in accuracy is minor and the simplification achieved is considerable if the shear allowance is neglected. ACI (2002) and CEN (2003) consider this approximation acceptable whereas NZS (1995) suggests the expression with respect to centre of joint.

ACI (2002) recommends that the sum of the nominal flexural strengths of the column section above and below the joint calculated at the joint faces using the factored axial load that results in the minimum column-flexural strength, should not be less than 1.2 times the nominal flexural strength of the beam sections at the joint faces. In T-beam construction, where the slab is in tension under moments at the face of the joint, slab reinforcement within an effective slab width suggested by the code should be assumed to contribute to the flexural strength of the beams, if the slab reinforcement is developed at the joint face for flexure.

As per the capacity design of columns for flexure, NZS (1995) requires the design flexural strength of the column to be in excess to the flexural overstrength of adjacent beams. With reference to the centre of the joint, this requirement is given as

$$\sum M_{nc} \geq 1.4 \sum \phi_i M_{nb}$$  \hspace{1cm} (2.2)
where \( \phi_o \) is overstrength factor for beams, which may be taken as 1.47 from the overstrength of steel as 1.25 and a strength reduction factor of 0.85. The column moments derived for lateral static forces are likely to be magnified by the participation of higher modes; and this effect is represented by dynamic moment magnification factor, the value of which ranges from 1.3 to 1.8 for one-way frames (Paulay and Priestley, 1992). The code includes this effect by adopting a value 1.4 as given in Equation (2.2). Hence it can be understood that the design flexural strength of columns are expected to be at least 2.06 times higher than the design flexural strength of adjoining beams. This factor is much greater than those suggested by CEN (2003) and ACI (2002).

CEN (2003) suggests the following condition to be satisfied at all joints:

\[
\sum M_{Re} \geq 1.3 \sum M_{Rb}
\]  

(2.3)

Where \( \sum M_{Re} \) is the sum of the design values of the minimum moments of resistance of the columns within the range of column axial forces produced by the seismic design situation and \( \sum M_{Rb} \) is the sum of the design values of the moment of resistance of the beams framing into the joint.

Dooley and Bracci (2001) derived probability curves for evaluation of column to beam strength ratio for each storey level in two case studies RC frames. The curves shown in Figure (2.5) can be broken down into three main regions: both the leftmost region and the rightmost region have only slight negative slopes, whereas the middle region has a large negative slope. Only within the middle region does increasing the column-to-beam strength ratio substantially improve performance. The last column-to-beam strength ratio belonging to the middle region could be considered an optimum ratio because increasing the strength ratio up to this point is very beneficial; however, increasing beyond this number does not provide significant benefits. For all four sets of data, the optimum column-to-beam strength ratio fell near 2.0 or greater. The probability of the formation of a storey mechanism with a column-to-beam strength ratio of 1.2, which is the minimum value required by ACI (2002), was roughly 90% in all four cases, which would generally be considered unacceptably high.
In terms of seismic performance, the research work concluded that increasing the column-to-beam strength ratio by increasing the column reinforcement ratio (strength alone) was more effective than increasing the column size (both strength and stiffness).

Ono et al. (2000) introduced a probabilistic evaluation using stochastic limit analysis of frames of the target values of the column overdesign factor (COF) for which the occurrence probability of un-preferable failure modes under both uncertain loads and member strengths can be limited to within a specific tolerance.

The research suggested that to ensure that the frame structures collapse according to the beam-failure pattern, a considerably larger value of COF is required in the case where both the loads and the member strengths are uncertain than in the case of a deterministic member strength. Additionally, for a given number of storeys, the larger the number of bays, the larger will be the target value of COF. Moreover, for a given number of bays, the larger the number of storeys, the larger will be the target value of COF. The target value of COF is more sensitive to the number of bays than to the number of storeys. Finally, the higher the reliability level is set when the structure is designed, the smaller the required target value of COF to limit the probability ratio to within a given tolerance level.

Figure 2.5: Probability of demand exceeding storey mechanism capacity versus strength ratio for each floor of three storey building (After Dooley and Bracci 2001).
In a recent research work by Zhao et al. (2006), the concept of the basic and optimum column overdesign factor (COF) to avoid storey mechanism according to a given reliability index is introduced. The basic COF is the lowest limit to ensure the entire beam-hinging mode formed prior to the storey collapse modes probabilistically. For a structure designed with a COF less than basic COF, the plastic hinges are easier to occur in partial storeys to form storey mechanism. The optimum COF is the most efficient COF to avoid the storey collapse modes. The storey collapse modes can be avoided effectively if the structure is designed with the COF between the basic COF and the optimum COF. Both factors decrease with increasing the reliability index of entire beam hinging mode.

![Figure 2.6: The basic and optimum COF to avoid storey mechanism (After Zhao et al. 2006)](image)

Lee (1996) conducted a study to investigate the effectiveness of the conventional capacity design rule for preventing columns hinging. The researcher observed that the distribution of bending moment in columns at some instants of time during time history analysis is far from the assumption that the point of contraflexure are located generally close to midheight of column while beams reach the inelastic range. The reason for that is the sudden change in moment’s distribution due to formation of many plastic hinges produce incremental moments that can be very large at some joints. After the formation of plastic hinges in beams, large moments are developed at top of columns in contrast to the small moments at bottom of columns above the joints. The research observed that the ratios of the moment of the lower column to that of the upper column are generally about 3 at the stage of incipient collapse. This means that three quarters of the sum of beams plastic moments is taken by the lower column. Lee suggested a three-quarter rule for strong-column and weak-beam design as follows:

\[ RM_p^c \geq 0.75\alpha \sum M_p^b \]  

(2.4)

Where \( \alpha \) = deformation hardening factor which is approximately equal 1.1; \( \sum M_p^b \) = sum of plastic moments of beams; \( \sum M_p^c \) = plastic moment of lower column; and \( R \) = reduction factor due to effect of axial force.
2.4 Prediction of failure mechanisms of capacity designed RC buildings

Response mechanism prediction of framed structures is a basic procedure used in the evaluation of seismic performance and capacity. The simulation of the failure modes and the prediction of the collapse of buildings under earthquake are very useful for studying the safety of buildings and evaluating losses during earthquakes. In deterministic design of framed structures, some preferred response modes are often selected and the strengths of members are assigned according to the strength requirements of the response mode. In reality, however, the designed structure may collapse unexpectedly according to some undesirable failure modes due to uncertainties in member strength and external load. Since it is impossible to absolutely ensure the structure responds according to the designed response mode in a deterministic meaning, it is essential to identify the likely failure modes and understand the order in which they are likely to occur.

Many studies on the collapse modes of frames have been performed so far, and basically either a deterministic or probabilistic method was employed in the analysis. In a recent research work by Zhao et al. (2007), the preferable and desirable failure modes of frame structures were investigated, and a focus study on the storey mechanism which is the failure pattern that should be avoided during seismic excitation was carried out. The study investigated the probabilistic order of storey mechanisms of frames designed under a certain reliability level.

The failure modes can be defined generally in three types:

1) Preferable mode, i.e., the strictly entire beam hinging pattern.
2) Allowable modes, including the nearly entire beam hinging pattern and entire collapse pattern.
3) Unallowable modes, i.e., the storey mechanism.
When the investigation is focused on storey mechanisms, there are generally \((2^n-1)\) storey mechanisms for an \(n\)-storey structure, including the entire beam hinging mode. The storey failure modes are classified into three patterns: upper collapse pattern, middle collapse pattern, lower collapse pattern as shown in Figure (2.8).

Upper collapse pattern is characterized by the continuous collapsed storeys from the top storey; lower collapse pattern is characterized by continuous collapse of storeys from first...
storey; middle pattern is with collapse of storeys in the middle with top and bottom storeys unyielding. Recent earthquake events have revealed a class of structures which can be referred to as mid-broken structures following the middle collapse pattern. As a definition, a mid-broken structure has a storey weakened by some kind of change in the material used or changes in geometry of the member’s cross sections. Until present, only very limited research has been dictated to this phenomenon. As concluded by Skjaerbaek et al. (1997) after series of shake table tests, the reason for failure in the weak storey is the low absolute stiffness in that storey which leads to large plastic deformations, thus, failure occurs when stiffness reaches a critical value and the storey is no longer able to transfer the shear forces from the storeys above.

Zhao et al. (2007) developed probability curves of each failure mode as a function of the COF, considering that the value of COF is uniformly distributed for all storeys and that the value of the COF is at least equal to 1.0 without considering the type of building that have strong beams and weak column pattern that should be avoided through the application of capacity design principles.

![Mode 1](image1) ![Mode 2](image2) ![Mode 3](image3) ![Mode 4](image4) ![Mode 5](image5) ![Mode 6](image6)

**Figure 2.9 : Probability of failure versus COF for middle storey collapse pattern (Zhao et al. 2007).**

Zhao et al. (2007) concluded that for the middle storey mechanisms, the plastic hinges are more likely to develop in lower storeys. The probability of the lower collapse mode is always higher than for middle collapse mode. The probability orders of lower storey mechanisms are affected notably by the COF values of frames, thus, for low-level COF, the plastic hinges tend to develop in a reduced number of storeys, and for high level COF, the hinging pattern tend to develop in a larger number of storeys.
2.5 Failure mechanism control

The mechanisms formed by strong columns and weak beams are now strongly recommended as a design concept for multi-storey frame structures, because energy dissipation occurs in plastic hinges at both ends of many beams during a major seismic event. However, as explained by Nagae et al. (2005), it is not easy to demonstrate quantitatively the advantages for the performance of the building, such as safety against collapse, even if the complete mechanism is guaranteed during an event. From the viewpoint of restoration, it is generally difficult to repair all hinges in many beams. In addition, costs of rehabilitation can be high because all of storeys of the building become the target of repair.

![Figure 2.10: Complete and partial mechanism of RC frames (Nagae et al. 2005).](image)

The type of mechanism of a structure during earthquakes can be controlled by strength ratios of the upper part to the lower part, i.e. by relatively strengthening the members in the upper part of the building. If a mechanism is located in a limited area, the rest of the building remains intact. Thus, repair work can be rationalized. The type of mechanism can be an option for performance based design, which should follow diverse demands from stakeholders or residents. On the other hand, it is obvious that a localized partial mechanism means the decrease in the number of members consuming energy. Thus, for a building designed based on mechanism control, it becomes very important to demonstrate adequate performances in terms of the damage process and the collapse capacity.

Nagae et al. (2005) demonstrates procedures to evaluate performance of RC frame structures with partial mechanism and considers the effects of the size of partial mechanism on the seismic performance. The considered partial mechanisms and summary results are shown in Figure (2.11). The analysis using incremental dynamic analysis showed that the sum of the hysteretic energy dissipations of all hinges and the sum of the maximum plastic rotations of all hinges at a given hazard level tend to be the same regardless of the size of the partial mechanism, respectively. Additionally, the tendency of maximum inter-storey drift with different extents of mechanisms can be explained as the partial mechanisms are enlarged by three storeys, the effect of the extent of partial mechanism decreases gradually. It is concluded that the energy dissipation concentrates in the lower area and is not being shared equally across the mechanism as the number of storeys becomes large.
The probability of collapse is reduced as the mechanism extent becomes large. However, the collapse probability decreases only slightly as the extent of the partial mechanism grows beyond four storeys.

Figure 2.11: Effect of mechanism control on RC frames response (after Nagae et al. 2005)

Qazi et al. (2007) conducted a critical review of the current preferable seismic failure mechanism. He showed that to insure the formation of beam sway mechanism, the deformations at the base of the first storey columns must be excessive to initiate the beam sway (Paulay and Priestley 1992). Therefore the formation of plastic hinges at the base of the first storey columns is inevitable. Although in some instances the formation of plastic hinges at the column bases may not be so critical regarding the safety of the structure, but it requires extensive rehabilitation efforts. Moreover the frame does not possess the re-centering ability after undergoing severe lateral drift during strong shaking and the chances of complete demolition of the structure are always there in case of excessive yielding at the column base sections. Furthermore, the possibility of exceeding the moment capacity at the top of columns still exists, and the soft first storey sway failure mechanism can be formed.

With the application of high performance materials in seismic design, Qazi et al. (2007) suggested a passive control practice by using high strength reinforcement steel in columns.
The mechanism of the passive control RC frames and its expected potential benefits against earthquakes has been compared with the ordinary RC frame.

The research work concluded that the new method prevent soft storey failure mechanism and provide more increased lateral load resistance capacity with less reparable cost by simple replacement of ordinary conventional steel in the frame columns with high tensile strength steel. Additionally, reduce the residual deformations and reduce the chances of complete demolition as a result of excessive yielding at column base sections. Moreover, the strengthening and rehabilitation demands are reduced on beam end sections.

![Figure 2.12: Reduction of residual deformation using high strength reinforcing steel in column sections (after Qazi et al.2006)](image1)

![Figure 2.13: Reduction of global structural damage using high strength reinforcing steel in column sections (after Qazi et al.2006)](image2)
3 CASE STUDY BUILDING FRAMES AND MODELLING

Seven structural models were used in the present study, chosen in order to cover various levels of irregularity in elevation, structural ductility, and in-plane stiffness. They are also intended to represent buildings with and without seismic design provisions, hence coverage of both old existing and modern structures is provided. The frames used for this study cover a wide range of variation in characteristics to help in deriving some generally - or widely - applicable conclusions. They have the following attributes:

- Frames designed for only gravity load resistance.
- Frames with high irregularities in strength and stiffness.
- Frame with embedded wide beams.
- Frames designed with seismic provisions.

For each case study, the geometrical properties, material strengths, and reinforcement details are presented. Additionally, the vertical distributions of the strength and stiffness are shown according to the sectional analyses for frame members which are tabulated in Appendix-A.

Whereas they are specific structures, conclusions from their response may be considered sufficiently indicative of the response of a much larger class of building.

3.1 Case Study-1: Italian 3 storey frame

Moratti (2000) described the design of a frame to just gravity loads according to the Italian specifications of the 1970’s. It has three-storeys and three bays and has been built to 2/3 scale at the Department of Structural Mechanics at the University of Pavia, Italy.

The geometrical and reinforcement characteristics of the three-storey, three bay frame are illustrated in Figure (3.1). The building was designed to sustain only gravity loads and therefore has some characteristics that differ from those of regular buildings built by seismic design codes. These characteristics cause deficiencies in structural response under earthquake loadings and thus should be considered carefully in the analytical assessment. Featuring the old practice of frames designed only for gravity loads, no transverse reinforcement were placed in the joint region. Smooth steel bars, with mechanical properties similar to those typically used in older periods, were adopted for both longitudinal and transverse
reinforcement. Beam bars in exterior joints were not bent in the joint region, but anchored with end-hooks. Lap splices with hook anchorages were adopted in the beam bars crossing interior joints as well as in column longitudinal bars at each floor level above the joint region and at the column-to-foundation connection.

Figure (3.2) shows the vertical distribution of strength and stiffness for the case study frame. The vertical strength distribution represents each storey shear capacity which is calculated according to the sectional analyses (see Chapter 4) of the frame members. The storey shear capacity is the sum of the shear strength of all columns developed at the flexural strength of column ends. The vertical distribution of stiffness is represented by the sum of gross stiffnesses of all columns at each storey as well as the sum of the effective stiffnesses which are calculated according to the sectional analyses results. Table (3.1) shows the material strength and geometric properties. Table (3.2) shows the mass distribution.

Figure 3.1 : Italian 3 storey frame geometry and reinforcement details.
Table 3.1: Material strength and geometric properties of the Italian 3 storey frame

<table>
<thead>
<tr>
<th>Material strength</th>
<th>Geometry</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete compression strength</td>
<td>17 MPa Storeys</td>
</tr>
<tr>
<td>Reinforcing Steel yield strength</td>
<td>370 MPa Typical storey height</td>
</tr>
<tr>
<td>Reinforcing Steel ultimate strength</td>
<td>470 MPa Total height</td>
</tr>
</tbody>
</table>

Table 3.2: Italian 3 storey frame mass distribution

<table>
<thead>
<tr>
<th>Floor level</th>
<th>Mass (Tonnes)</th>
<th>Weight (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>5.52</td>
<td>54.15</td>
</tr>
<tr>
<td>2</td>
<td>7.44</td>
<td>72.98</td>
</tr>
<tr>
<td>1</td>
<td>7.44</td>
<td>72.98</td>
</tr>
<tr>
<td>Total weight</td>
<td></td>
<td>200.12 KN</td>
</tr>
</tbody>
</table>

Figure 3.2: Italian 3 storey frame stiffness and strength distribution.

3.2 Case Study-2: Italian 6 storey frame

Moratti (2000) described the design of a frame to just gravity loads according to the Italian specifications of the 1970’s (see Figure 3.3). It has six storeys and three bays and has the characteristics of vertical irregularities in strength and stiffness with height, as can be seen in Figure (3.4), which is a common practice for gravity load designed frames without seismic provisions. Table (3.3) shows the material strength and geometric properties. Table (3.4) shows the mass distribution.
Chapter 3. Case Studies Building Frames and Modelling

Frame Geometry

FE modelling and mass distribution

Reinforcement Details

Figure 3.3: Italian 6 storey frame geometry and reinforcement details.

Table 3.3: Material strength and geometric properties of the Italian 6 storey frame

<table>
<thead>
<tr>
<th>Material strength</th>
<th>Geometry</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete compression strength</td>
<td>Storeys</td>
<td>6</td>
</tr>
<tr>
<td>Reinforcing Steel yield strength</td>
<td>Typical storey height</td>
<td>3.0 m</td>
</tr>
<tr>
<td>Reinforcing Steel ultimate strength</td>
<td>Total height</td>
<td>18.0 m</td>
</tr>
</tbody>
</table>
### Chapter 3. Case Studies Building Frames and Modelling

#### Table 3.4: Italian 6 storey frame mass distribution

<table>
<thead>
<tr>
<th>Floor level</th>
<th>Mass (Tonnes)</th>
<th>Weight (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>25.25</td>
<td>247.7</td>
</tr>
<tr>
<td>5</td>
<td>30.25</td>
<td>296.7</td>
</tr>
<tr>
<td>4</td>
<td>30.51</td>
<td>299.2</td>
</tr>
<tr>
<td>3</td>
<td>29.95</td>
<td>293.8</td>
</tr>
<tr>
<td>2</td>
<td>30.51</td>
<td>299.3</td>
</tr>
<tr>
<td>1</td>
<td>30.16</td>
<td>295.9</td>
</tr>
<tr>
<td><strong>Total weight</strong></td>
<td><strong>1732.7 KN</strong></td>
<td></td>
</tr>
</tbody>
</table>

#### Figure 3.4: Italian 6 storey frame stiffness and strength distribution.

#### 3.3 Case Study-3: ICONS 4 storey frame

The third case study shown in Figure (3.5) is a four-storey bare frame known to have a soft storey at the third floor due to deficiencies in the design. The latter is attributed to a drastic change in strength and stiffness at this level through a reduction in both the reinforcement content and the section dimensions in the columns between the second and third storeys (see Figure 3.6). Such characteristics are common in buildings designed predominantly for gravity loads. Table (3.5) shows the material strengths and geometric properties. Table (3.6) shows the mass distribution.

The reinforced concrete bare frame was designed and built at the European Laboratory for Structural Assessment (ELSA) of the Joint Research Centre (JRC) at Ispra, Italy. The full-scale model was constructed for pseudo-dynamic testing, under the auspices of the EU funded ECOEST/ICONS programme (Carvalho et al., 1999). The frame was designed essentially for gravity loads and a nominal lateral load of 8% of its weight. The reinforcement details attempted to mirror the construction practices used in southern European countries in the 1950’s and 1960’s. The columns were non ductile, smooth reinforcing bars were used, capacity design principles were ignored and lap splicing occurred in critical regions.
Chapter 3. Case Studies Building Frames and Modelling

Figure 3.5: ICONS 4 storey frame geometry and reinforcement details.
Table 3.5: Material strength and geometric properties of ICONS 4 storey frame

<table>
<thead>
<tr>
<th>Material property</th>
<th>Value</th>
<th>Geometry</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete compression strength</td>
<td>16.3 MPa</td>
<td>Storeys</td>
<td>4</td>
</tr>
<tr>
<td>Reinforcing Steel yield strength</td>
<td>343 MPa</td>
<td>Typical storey height</td>
<td>2.7</td>
</tr>
<tr>
<td>Reinforcing Steel ultimate strength</td>
<td>452 MPa</td>
<td>Total height</td>
<td>10.8m</td>
</tr>
</tbody>
</table>

Table 3.6: ICONS 4 storey frame mass distribution.

<table>
<thead>
<tr>
<th>Floor level</th>
<th>Mass (Tonnes)</th>
<th>Weight (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>37.50</td>
<td>367.88</td>
</tr>
<tr>
<td>3</td>
<td>45.50</td>
<td>446.36</td>
</tr>
<tr>
<td>2</td>
<td>45.50</td>
<td>446.36</td>
</tr>
<tr>
<td>1</td>
<td>45.50</td>
<td>446.36</td>
</tr>
</tbody>
</table>

Total weight: 1706.94 KN

Figure 3.6: ICONS 4 storey frame stiffness and strength distribution.

3.4 Case Study-4: Turkish 5 storey pilotis frame

The fourth case study shown in Figure (3.7) is a five-storey bare frame known to have a soft storey with a taller first floor where there exists a stiffness irregularity as shown in Figure (3.8). This frame represents a common practice of RC buildings in Turkey that is designed only for gravity loads without seismic provisions. The frame is extracted from a Master thesis (Basaran, 2006). Table (3.7) shows the material strength and geometric properties. Table (3.8) shows the mass distribution.
Chapter 3. Case Studies Building Frames and Modelling

Frame Geometry

FE modelling and mass distribution

Reinforcement Details

Columns Sections

<table>
<thead>
<tr>
<th>Column</th>
<th>Section</th>
<th>Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>0.3mx0.3m</td>
<td>8Ø14mm</td>
</tr>
<tr>
<td>C2</td>
<td>0.4mx0.4m</td>
<td>8Ø16mm</td>
</tr>
</tbody>
</table>

Beams Sections

1st, 2nd, 3rd, 4th Floor beams

5th Floor beams

Figure 3.7: Turkish 5 storey pilotis frame geometry and reinforcement details.
Table 3.7: Material strength and geometric properties of the Turkish 5 storey pilotis frame

<table>
<thead>
<tr>
<th>Material strength</th>
<th>Geometry</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete compression strength 17 MPa</td>
<td>Storeys 5</td>
</tr>
<tr>
<td>Reinforcing Steel yield strength 371 MPa</td>
<td>Typical storey height 3.0 m</td>
</tr>
<tr>
<td>Reinforcing Steel ultimate strength 470 MPa</td>
<td>Total height 16.5 m</td>
</tr>
</tbody>
</table>

Table 3.8: Turkish 5 storey pilotis frame mass distribution

<table>
<thead>
<tr>
<th>Floor level</th>
<th>Mass (Tonnes)</th>
<th>Weight (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>20.9</td>
<td>204.6</td>
</tr>
<tr>
<td>4</td>
<td>37.6</td>
<td>369.2</td>
</tr>
<tr>
<td>3</td>
<td>37.6</td>
<td>369.2</td>
</tr>
<tr>
<td>2</td>
<td>37.6</td>
<td>369.2</td>
</tr>
<tr>
<td>1</td>
<td>37.6</td>
<td>369.2</td>
</tr>
<tr>
<td><strong>Total Weight</strong></td>
<td><strong>1681.3 KN</strong></td>
<td></td>
</tr>
</tbody>
</table>

3.5 Case Study-5: Turkish 7 storey embedded beams frame

The fifth case study shown in Figure (3.9) represents a typical residential-commercial building from the Bekirpasa district of Kocaeli, Turkey (Bal et al. 2006). This frame is a reinforced concrete moment-resisting frame with wide beam–column connections and one-way joists slabs which have been in common use in countries of the moderate to high-seismicity Mediterranean area, such as Italy and Turkey, despite the lack of sufficient information on how the system behaves under severe earthquake loading. Figure (3.10) shows the vertical distribution of strength and stiffness. Table (3.9) shows the material strength and geometric properties. Table (3.10) shows the mass distribution.
Figure 3.9: Turkish 7 storey embedded beams frame geometry and reinforcement details.
Table 3.9: Material strength and geometric properties of the 7 storey Turkish embedded beam frame

<table>
<thead>
<tr>
<th>Material strength</th>
<th>Geometry</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete compression strength 16.7 MPa</td>
<td>Storeys 7</td>
</tr>
<tr>
<td>Reinforcing Steel yield strength 371 MPa</td>
<td>Typical storey height</td>
</tr>
<tr>
<td>Reinforcing Steel ultimate strength 470 MPa</td>
<td>Total height 18.2m</td>
</tr>
</tbody>
</table>

Table 3.10: Turkish 7 storey embedded beams frame mass distribution

<table>
<thead>
<tr>
<th>Floor level</th>
<th>Mass (Tonnes)</th>
<th>Weight (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>76.52</td>
<td>750.66</td>
</tr>
<tr>
<td>6</td>
<td>76.52</td>
<td>750.66</td>
</tr>
<tr>
<td>5</td>
<td>76.52</td>
<td>750.66</td>
</tr>
<tr>
<td>4</td>
<td>76.52</td>
<td>750.66</td>
</tr>
<tr>
<td>3</td>
<td>76.52</td>
<td>750.66</td>
</tr>
<tr>
<td>2</td>
<td>76.52</td>
<td>750.66</td>
</tr>
<tr>
<td>1</td>
<td>76.52</td>
<td>750.66</td>
</tr>
<tr>
<td>Total Weight</td>
<td>5254.62</td>
<td>2002.30</td>
</tr>
</tbody>
</table>

Figure 3.10: Turkish 7 storey embedded beams stiffness and strength distribution.

Since wide beam–column connections (see Figure 3.11) are less widely used than conventional beam–column connections, there is very little specific research into their seismic behaviour (Climent, 2005). The major drawbacks of this structural system are the low lateral stiffness, the deficient transmission of bending moments from the wide beams to the columns and the poor energy dissipation capacity. Because of these drawbacks, most design codes currently prevent or restrict the use of wide beam systems in seismic regions. The need to evaluate the vulnerability of such existing structures is increasingly understood by both structural engineers and building owners.
3.6 Case Study-6: Yugoslavian 7 storey capacity designed frame

The sixth case study frame shown in Figure (3.12) is extracted from a full-scale seven storey RC building that has been designed and built through the US-Japan Cooperative earthquake Engineering Research Program (ACI, 1984). Four designs were made to various codes and the design chosen to be modeled for this study was to the Yugoslavian code of 1981, hence it can be considered a European building designed for an area of high seismicity. Figure (3.13) shows the vertical distribution of strength and stiffness. Table (3.11) shows the material strength and geometric properties. Table (3.12) shows the mass distribution.

<table>
<thead>
<tr>
<th>Material strength</th>
<th>Geometry</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete compression strength 29 MPa</td>
<td>Storeys 7</td>
</tr>
<tr>
<td>Reinforcing Steel yield strength 372 MPa</td>
<td>Typical storey height 3.0</td>
</tr>
<tr>
<td>Reinforcing Steel ultimate strength 470 MPa</td>
<td>Total height 21.75m</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Floor level</th>
<th>Mass (Tonnes)</th>
<th>Weight (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>58.14</td>
<td>570.35</td>
</tr>
<tr>
<td>6</td>
<td>65.28</td>
<td>640.39</td>
</tr>
<tr>
<td>5</td>
<td>65.28</td>
<td>640.39</td>
</tr>
<tr>
<td>4</td>
<td>65.28</td>
<td>640.39</td>
</tr>
<tr>
<td>3</td>
<td>65.28</td>
<td>640.39</td>
</tr>
<tr>
<td>2</td>
<td>65.28</td>
<td>640.39</td>
</tr>
<tr>
<td>1</td>
<td>70.38</td>
<td>690.42</td>
</tr>
<tr>
<td></td>
<td>Total Weight</td>
<td>4462.76 KN</td>
</tr>
</tbody>
</table>
Figure 3.12: Yugoslavian 7 storey frame geometry and reinforcement details.
3.7 Case Study-7: Portuguese 4 storey capacity designed frame

The seventh case study shown in Figure (3.14) represents a RC frame designed according to the Portuguese seismic code by Romao (2002). The frame is designed following the capacity design rules that insure beam hinging side-sway pattern. Figure (3.15) shows the vertical distribution of strength and stiffness. Table (3.13) shows the material strength and geometric properties. Table (3.14) shows the mass distribution.

Table 3.13: Material strength and geometric properties of the Portuguese 4 storey frame

<table>
<thead>
<tr>
<th>Material strength</th>
<th>Geometry</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete compression strength</td>
<td>28 MPa Storeys 4</td>
</tr>
<tr>
<td>Reinforcing Steel yield strength</td>
<td>460 MPa Typical storey height 3.0</td>
</tr>
<tr>
<td>Reinforcing Steel ultimate strength</td>
<td>560 MPa Total height 12 m</td>
</tr>
</tbody>
</table>

Table 3.14: Portuguese 4 storey frame mass distribution

<table>
<thead>
<tr>
<th>Floor level</th>
<th>Mass (Tonnes)</th>
<th>Weight (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>51.10</td>
<td>501.29</td>
</tr>
<tr>
<td>3</td>
<td>71.60</td>
<td>702.40</td>
</tr>
<tr>
<td>2</td>
<td>71.60</td>
<td>702.40</td>
</tr>
<tr>
<td>1</td>
<td>71.60</td>
<td>702.40</td>
</tr>
<tr>
<td><strong>Total Weight</strong></td>
<td><strong>2608.48 KN</strong></td>
<td></td>
</tr>
</tbody>
</table>
Figure 3.14: Portuguese 4 storey frame geometry and reinforcement details.
3.8 Numerical Modelling Software

3.8.1 Introduction to modelling of reinforced concrete frames

Many models for the non-linear analysis of RC frames are proposed in the literature. They can be classified depending on the level of discretization in point by point model, member by member models and global models. The choice of the most suitable model depends on the goals of the analysis and on the structural properties. Structures characterized by brittle mechanisms require a non-linear analysis and the use of highly discretized models; on the other hand, for structures with flexural collapse mechanisms, member by member or global models can be used to obtain reliable predictions. A good balance between computational effort and level of reliability of the results should be achieved in choosing the model by taking into account the amount of basic information that each model requires.

In the last few years, fibre models have become more and more popular. They still keep the basic hypothesis of subdividing the structure in mono-dimensional elements, even though they could be defined as a hybrid between point by point and member by member models. The main mechanisms influencing the behaviour of reinforced concrete frames are summarized in Figure (3.16).
Figure 3.16: The main mechanisms influencing the behaviour of reinforced concrete frames (after Cosenza 2006).

The finite element analysis package SeismoStruct (SeismoSoft, 2008) is utilized to perform the necessary analyses for the assessment of the test models such as nonlinear static pushover analysis, eigenvalue analysis and nonlinear dynamic response history analysis. The program is capable of representing spread of inelasticity within the member cross-section and along the member length utilizing the fiber analysis approach. SeismoStruct can be used to predict the behaviour of frames under static or dynamic loading, taking into account both geometric and material nonlinear behaviour. Accurate concrete and steel material models are available, together with a large library of three dimensional elements that can be used with a wide choice of steel, concrete and composite section configurations. The applied loading can be constant or variable forces, displacements and accelerations.

3.8.2 Material Modelling

The Mander et al (1988) constant confinement material model (see Figure 3.17) was used for concrete. This requires the input of concrete compressive strength ($f'_c$), tensile strength ($f_t$), strain at peak stress and a confinement factor. The confinement factor is defined as the ratio between the confined and unconfined compressive stress of the concrete, and is used to scale up the stress-strain relationship throughout the entire strain range. This material model was used for both confined and unconfined concrete, with the confinement factor for the latter being taken as 1.0. When information regarding the tensile strength and strain at peak stress was not available, the former has been calculated as $0.5\sqrt{f'_c}$ and the latter taken as 0.002.
Figure 3.17: Mander et al (1988) concrete model in SeismoStruct (SeismoSoft, 2008)

The bi-linear stress strain with strain hardening model was used for steel (Figure 3.18). This model requires the definition of yield strength ($f_y$), modulus of elasticity ($E$), and a strain hardening parameter. The strain hardening parameter is the ratio between the post-yield stiffness ($E_{sp}$) of the material and the initial elastic stiffness ($E_s$). The former is defined as $E_{sp} = (f_{ult} - f_y) / \left( \frac{\sum_{ult} - f_y}{E_s} \right)$, where $f_{ult}$ and $\sum_{ult}$ represent the ultimate or maximum stress and strain capacity of the material, respectively. When information was not available for the modulus of elasticity and strain hardening parameter, the former was assumed to be $2 \times 10^5$ MPa and the latter to be 0.005.

Figure 3.18: Bilinear stress-strain steel model with strain hardening, SeismoStruct (SeismoSoft, 2008)

3.8.3 Element Formulation

To allow for accurate estimation of structural damage distribution, the spread of material inelasticity along the member length and across the section area is explicitly represented through the employment of a fibre modelling approach, implicit in the formulation of SeismoStruct's inelastic beam-column frame elements. The sectional stress-strain state of beam-column elements is obtained through the integration of the nonlinear uniaxial stress-strain response of the individual fibres in which the section has been subdivided. The discretisation of a typical reinforced concrete cross-section is depicted, as an example, in
Figure (3.19). If a sufficient number of fibres (200-400 in spatial analysis) are employed, the distribution of material nonlinearity across the section area is accurately modelled, even in the highly inelastic range. Two integration Gauss points per element are then used for the numerical integration of the governing equations of the cubic formulation (stress/strain results in SeismoStruct always refer to these Gauss Sections, not to the element end-nodes). In all the structural models, 4 to 5 elements, with smaller elements at member ends, were used to model beams and columns to ensure inelasticity could be accurately modelled.

Rigid-end zones at the beam-column joints were used. This assumption is often used for structural analysis of reinforced concrete structures. As shown in Figure (3.20), rigid elements were placed at every beam column joint. This prevents plastic hinges from forming inside the joints and moves the inelastic behaviour outside the joint region where it is expected to occur. Table (3.15) shows a summary of the numerical modelling assumptions.

**Figure 3.19 : Discretisation of an RC section into fibers (SeismoStruct, 2008)**

**Figure 3.20 : Definition of rigid joints (after Bai and Hueste 2007).**
### Table 3.15: Summary table for numerical modelling assumptions

<table>
<thead>
<tr>
<th>Items in the numerical model</th>
<th>Assumption</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Material</strong></td>
<td></td>
</tr>
<tr>
<td>Reinforcement steel</td>
<td>Bilinear Elasto-plastic model with strain hardening</td>
</tr>
<tr>
<td>Concrete</td>
<td>The Mander et al (1988) constant confinement material model.</td>
</tr>
<tr>
<td></td>
<td>Confinement factor $K = 1.0$ (Unconfined part of the section)</td>
</tr>
<tr>
<td></td>
<td>$K = 1.1$ (Confined part of the section for older buildings)</td>
</tr>
<tr>
<td></td>
<td>$K = 1.4$ (Confined part of the section for newer buildings)</td>
</tr>
<tr>
<td>Loading</td>
<td></td>
</tr>
<tr>
<td>Self weight of RC members</td>
<td>25000 kg/m³</td>
</tr>
<tr>
<td>Gravity loads</td>
<td>$DL + 0.3LL$</td>
</tr>
<tr>
<td>Seismic dead load for mass</td>
<td>$DL + 0.3LL$</td>
</tr>
<tr>
<td>calculation</td>
<td></td>
</tr>
<tr>
<td>Mass distribution</td>
<td>Lumped at beam column connections</td>
</tr>
<tr>
<td>P-delta effect</td>
<td>Considered</td>
</tr>
<tr>
<td>Viscous damping</td>
<td>No (Only hysteretic damping was considered.)</td>
</tr>
<tr>
<td><strong>Structural modelling</strong></td>
<td></td>
</tr>
<tr>
<td>Analysis program</td>
<td>SeismoStruct</td>
</tr>
<tr>
<td>Element model</td>
<td>Distributed plasticity model</td>
</tr>
<tr>
<td>Center-line dimensions</td>
<td>Yes</td>
</tr>
<tr>
<td>Additional deformations at</td>
<td>Not considered</td>
</tr>
<tr>
<td>element intersections and</td>
<td></td>
</tr>
<tr>
<td>footing interface</td>
<td></td>
</tr>
<tr>
<td>M-M-N interaction</td>
<td>Yes</td>
</tr>
<tr>
<td>Beam and columns dimensions</td>
<td>Extended from centre of one beam-column joint to centre of the next one.</td>
</tr>
</tbody>
</table>
Chapter 4. Sway Potential Index

4 SWAY POTENTIAL INDEX

Evaluation of the most probable side-sway mechanism due to seismic excitation is considered as a basic step in seismic performance assessment of reinforced concrete frames. A reliable estimation of the side-sway behaviour of the structure leads to a better prediction of its seismic capacity. The development and formulation of a strength based as well as a deformation based sway potential indices for the prediction of the side-sway potential at each floor level of the building are discussed in this chapter along with their application to the case study bare frames described in the previous chapter. The strength based index relies on the degree of accuracy in estimating the available flexural strengths of the beams and the columns at critical sections at the frame joints. The deformation based index relies on the estimated yield rotation capacities of the beams and the columns which is a function of the geometry of the members.

4.1 Member Flexural Strengths

In the assessment of an existing structure, realistic values for the material strengths should be used to obtain the best estimate of probable strength of the members. It follows that the use of the nominal material strengths that were specified in the original design is inappropriate.

The probable flexural strengths of beam and column plastic hinges at each joint in the frame are calculated using the probable material strengths in the moment curvature section analysis program CUMBIA (2007) developed by L.Monteo and M.Kowalsky at North Carolina State University. A strength reduction factor $\phi = 1.0$ may be assumed for this flexural strength calculation since the probable properties of the members are used as built.

CUMBIA is a set of Matlab codes to perform monotonic moment-curvature analysis and force displacement response of reinforced concrete members of rectangular or circular sections. An axial load – moment interaction analysis can also be performed. The section analysis is performed by tabulating moment and curvature of the member section for increasing levels of concrete strain. The member response is obtained from the section moment-curvature results along with an equivalent plastic hinge length, as suggested by Priestley et al. (1996). The constitutive models for the concrete and steel can be easily specified by the user. The code allows the analysis of members subjected to axial load (tension or compression) and single or double bending.
4.1.1 Beam hinge flexural capacity

Beam flexural strength should be assessed by using an ultimate compression strain of 0.004, since this represents a lower limit on recorded crushing strains of plastic hinges forming against supporting members (in this case, columns). However, if a moment curvature analysis incorporating strain hardening of reinforcement is used to assess flexural strength, positive moment capacity predicted at an extreme fiber compression strain of 0.004 will correspond to excessive tensile strains, when, as normally the case, the area of the top (compression) reinforcement exceeds that of the bottom (tension). For such cases, flexural strength should be assessed when the peak tensile strain in reinforcement is about $\varepsilon_s = 0.02$. These limits are suggested by Priestley (1997).

4.1.2 Column hinge flexural capacity

The procedure outlined above also applies to plastic hinges forming at column bases, or in column side-sway mechanisms. It is noted that the column moment capacities depend on the axial forces in the columns, which in turn depends on the lateral forces, which are unknown at the start of the assessment process. It is normally only the outer columns that will be significantly affected, and it is also noted that unless the gravity axial force level is unusually high, the reduction in column moment capacity of the outer column subjected to axial tension will be almost exactly balanced by the increase in the moment capacity of the opposite outer column which is subjected to axial compression. This implies that for most cases, the sway index may be determined using column moments calculated for gravity axial loads without consideration of seismic axial forces (Priestley, 1997).

4.1.3 Flexural capacities of the members of case-study frames

A sample evaluation is shown in Tables (4.1) and (4.2) of the flexural strength capacities resulting from moment curvature analysis of beams and column potential plastic hinges of case study-1, the Italian three storey frame. The detailed sectional analysis of all case study frames are listed in Appendix-A.

<table>
<thead>
<tr>
<th>Beams</th>
<th>Reinforcement (mm$^2$)</th>
<th>Reinforcement Ratio %</th>
<th>Flexural Strength My (KN.m)</th>
<th>Tension side</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top</td>
<td>Bottom</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bay1</td>
<td>326.73</td>
<td>326.73</td>
<td>0.53</td>
<td>38.42</td>
</tr>
<tr>
<td></td>
<td>439.82</td>
<td>100.53</td>
<td>0.71</td>
<td>50.77</td>
</tr>
<tr>
<td>Bay2</td>
<td>439.82</td>
<td>100.53</td>
<td>0.16</td>
<td>12.33</td>
</tr>
<tr>
<td></td>
<td>326.73</td>
<td>213.63</td>
<td>0.53</td>
<td>38.14</td>
</tr>
<tr>
<td>Bay3</td>
<td>326.73</td>
<td>213.63</td>
<td>0.34</td>
<td>14.84</td>
</tr>
<tr>
<td></td>
<td>326.73</td>
<td>213.63</td>
<td>0.53</td>
<td>38.14</td>
</tr>
</tbody>
</table>

Table 4.1: Flexural capacities of beam sections for case study-1
Table 4.2: Flexural capacities of column sections for case study-1

<table>
<thead>
<tr>
<th>Column</th>
<th>Storey</th>
<th>Axial force (KN)</th>
<th>Axial Load Ratio %</th>
<th>Reinforcement (mm^2)</th>
<th>Reinforcement Ratio %</th>
<th>Flexural Strength My(KN.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>3</td>
<td>12.60</td>
<td>0.019</td>
<td>301.59</td>
<td>0.75</td>
<td>10.72</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>25.20</td>
<td>0.037</td>
<td>301.59</td>
<td>0.75</td>
<td>11.56</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>37.80</td>
<td>0.056</td>
<td>301.59</td>
<td>0.75</td>
<td>12.50</td>
</tr>
<tr>
<td>C2</td>
<td>3</td>
<td>18.18</td>
<td>0.027</td>
<td>301.59</td>
<td>0.75</td>
<td>11.10</td>
</tr>
<tr>
<td></td>
<td>2</td>
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4.2 Strength Based Sway Index

Having determined the probable flexural strengths of the members and joints of the frame, the next step in the assessment procedure is to identify the probable location of post-elastic deformations due to severe earthquake forces and hence to determine the critical mechanism of post-elastic deformation. This will involve determining whether flexural plastic hinges occur in the beams or the columns at each beam-column joint and/or whether shear failure occurs in the members or joints.

Often for a building frame the critical mechanism is not simply a beam side-sway mechanism or a column side-sway mechanism, but is a mixed mechanism involving flexural plastic hinges at some locations combined with shear failures of members and/or joints at other locations (see Figure 4.1).

![Figure 4.1: Possible mechanisms of post-elastic deformation of moment resisting frames (NZSEE, 2002)](image)
To investigate whether plastic hinges occur in beams or columns, a sway potential index $S_i$ can be defined as proposed by Priestley (1997) for the beam-column joints at a horizontal level by comparing the sum of the probable flexural strengths of the beams and the columns at the joint centroids:

$$S_i = \frac{\sum (M_{bl} + M_{br})}{\sum (M_{ca} + M_{cb})}$$

(4.1)

where $M_{bl}, M_{br}$ = beam probable flexural strengths at the left and right of the joint, respectively, at the joint centroid, and $M_{ca}$ and $M_{cb}$ are column flexural strengths above and below the joint, respectively, at the centroid of the joint. These are summed for all the joints at that horizontal level. RC frame buildings may undergo a storey mechanism or column failure mechanism in cases where plastic hinges form at the ends of all column members in a storey.

When $S_i > 1$, column plastic hinges can be expected. However, to include the effects of higher modes of vibration, and a possible overestimation of column flexural strength, it is suggested by Priestley (1997) that column plastic hinges form if $S_i > 0.85$. Accordingly, the dynamic magnification factor need not be applied in this procedure, however; considering the mid-rise RC buildings used in this research work, dynamic amplification is not needed to be introduced in this formula.

It is apparent that the formula does not take into account the effect of the stiffness of the members which is significant especially in the case of tall first storey buildings that could have sufficient flexural strength at column ends but reduced stiffness compared to the beams.

### 4.3 Deformation Based Sway Index

#### 4.3.1 Background

A newly proposed index is introduced herein to help in predicting the most probable side-sway mechanism of reinforced concrete frames considering the stiffness irregularities that exist at storey levels and the relative flexibility of beams and columns using only the geometrical properties as an indicator of the potential failure mechanism.

The proposed sway index follows the same final formulation of Priestley (1997) in comparing the capacities of the beams and the columns at each floor level; the only difference is that the comparison is done using the yield rotation rather than the flexural strength capacity of the members. The main advantage of using the yield rotation is that it is only a function of the section dimensions and member length.

For reinforced concrete buildings, the calculation of the yield rotation capacities of the beams and columns for the two mechanisms is based on simple principles of mechanics related to the behaviour of reinforced concrete members.
Priestley (1998) first showed how the yield curvature, $\phi_y$, of reinforced concrete sections is independent of the strength, but dependent on the yield strain of the reinforcement steel and the geometry of the section, as shown in the following equations:

For beams:
$$\phi_y = \frac{1.7 \varepsilon_y}{h_b} \quad (4.2)$$

For rectangular columns:
$$\phi_y = \frac{2.12 \varepsilon_y}{h_c} \quad (4.3)$$

where $h_b$ is the height of the beam section, $h_c$ is the depth of the column section and $\varepsilon_y$ is the yield strain of the reinforcement steel.

For beams and columns, the assumed curvature distribution varies linearly with a zero value at the mid-span of the member. Thus the beam and column yield curvatures presented can be used in conjunction with the assumed curvature distributions, which are integrated to predict the yield chord rotation of either the beam ($\theta_{by}$) the column ($\theta_{cy}$) as shown in the following equations:

For beams:
$$\theta_{by} = \phi_y \cdot L_b \cdot \frac{1}{2} \cdot \frac{1}{3} = 0.283 \varepsilon_y \cdot \frac{L_b}{h_b} \quad (4.4)$$

For rectangular columns:
$$\theta_{cy} = \phi_y \cdot h_c \cdot \frac{1}{2} \cdot \frac{1}{3} = 0.357 \varepsilon_y \cdot \frac{h_c}{h_c} \quad (4.5)$$

### 4.3.2 Relative flexibility of beams and columns

Applying the yield rotation formulas for beams and columns to the case study reinforced concrete frames, and comparing with the resulting flexural strength capacities, the usefulness of the relative comparison of yield rotations of beams and columns at frame joints at each floor level was observed. The yield rotation formula was found to be a good indicator of the relative flexibility of the concrete members, which is an important factor in controlling the expected side-sway mechanism of the frame.

The moment curvature analysis of the potential plastic hinge sections in the case study frames gives a starting point to use the yield rotation as a measure of side-sway mechanism. As shown in Figure (4.2), there is a trend of flexural strength reduction with increasing section yield curvature; they are inversely proportional. To relate the strength and stiffness of the concrete sections, Figure (4.3) shows a trend of the reduction of section yield curvature with increasing the gross sectional moment of inertia; these parameters are also inversely proportional.
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Figure 4.2: Relation between flexural strength and section yield curvature for the case study frames.

Figure 4.3: Relation between section yield curvature and gross moment of inertia for the case study frames.
A further step using the previous observations is to relate the gross stiffness of the concrete members to the flexural strength and to the members yield rotations as shown in Figures (4.4) and (4.5) respectively. A reduction in the member ends yield rotation occurs with increasing the flexural gross stiffness, whilst the flexural strength and gross stiffness are proportional, though there is scatter due to the fact that the actual moment capacity is proportional to the cracked stiffness of the member, not the gross stiffness. These relations can lead to a general observation that stiffer members have larger flexural strength capacity and lower yield rotation, in other words, less flexible members.

Considering the scattering in the plots, the beams have large scatter than the columns, which is expected from the difference between the negative and positive moment capacities of the beams for the same cross sectional dimensions and consequently the same yield rotation. However, as will be shown in the individual frame analysis results, the most important factor in defining the dominant side-sway mechanism is that the members that have larger yield rotation, and are consequently more flexible, are expected to control the behaviour of the frames under seismic excitation.

![Graph showing the relationship between moment capacity and gross stiffness](image)

**Figure 4.4**: Member’s flexural strength and gross stiffness for the case study frames.
Chapter 4. Sway Potential Index

Trend Line
\[ \theta_y = 0.063(EI/L)g^{0.28} \]
\[ R^2 = 0.682 \]

Figure 4.5: Member’s end yield rotations and gross stiffness for the case study frames.

In order to explain the previous observations, the moment-rotation relationships for selected beam-column joints for the case study frames are discussed. As a first example, the case study-4 (Turkish gravity load designed frame) is chosen and the moment-rotation curves for a beam-column joint at the first floor level are plotted in Figure (4.6). A large difference in the beams moment capacities is observed due to the difference in the positive and negative reinforcement ratios, but according to the formulation of the deformation based sway index, the average of the yield rotations of the columns will be larger than the average for the beams.

Figure 4.6: Moment-rotation curves for a beam-column joint of case study-4.
The same conclusion can be drawn from the second example shown in Figure (4.7) of case study-6 that represents a frame designed with seismic provisions. The capacity design principle is followed as the columns are stronger and less flexible. Additionally, the difference in the beam moments is smaller compared to the non-seismically designed frame in the previous example.

In order to use simple geometrical parameters to relate the flexibility of the beams and the columns to their strength, the relation between the members flexural strength and the ratio (section depth / member length) is plotted in Figure (4.8) which follows the expected trend as in Figure (4.4) that relates the flexural strength to the gross stiffness but with less correlation coefficient. It can be observed that the beams have larger scatter than the columns as discussed earlier. The reason behind that is because the gross stiffness is a parameter which is related to the section depth in 3rd order, while it is expected to use simple first order relation in the final formulation of the newly proposed deformation based sway index which is discussed in details in the next section.
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Figure 4.8: Member’s flexural strength and the ratio (section depth/member length) for the case study frames

4.3.3 Development of the deformation based sway index formula

The formulation of the deformation based sway index is introduced considering the results obtained from the moment curvature analysis of the members of the case study frames. The definition of the most probable side-sway mechanism of the frame is evaluated by comparing the yield rotation of the columns to those of the beams. As discussed in the previous section, the new index is a measure of the dominant flexible members that is expected to control the behaviour of the frame. Equation (4.5) introduces the basic formulation of the new index.

\[
R_t = \frac{Average \ \theta_{\text{column}}}{Average \ \theta_{\text{beams}}} = \frac{\left(\frac{\theta_{\text{cy,A}} + \theta_{\text{cy,B}}}{2}\right)}{\left(\frac{\theta_{\text{cy,L}} + \theta_{\text{cy,R}}}{2}\right)} \tag{4.5}
\]

where:
- \(\theta_{\text{cy,A}}\) is the yield rotation of the column above the joint
- \(\theta_{\text{cy,B}}\) is the yield rotation of the column below the joint
- \(\theta_{\text{cy,L}}\) is the yield rotation of the beam to the left of the joint
- \(\theta_{\text{cy,R}}\) is the yield rotation of the beam to the right of the joint
A correction has to be done to the rotations of the beam due to the effect of the column physical depth in the following equation:

$$\theta_{by} = \theta_{cy} \left( 1 + \frac{h_z}{L_{cb}} \right)$$  \hspace{1cm} (4.6)

Where $L_{cb}$ is the beam clear span between columns faces.

For a given joint, the comparison between the sum of the yield rotations of the upper and lower columns and the sum of the yield rotations of the left and right beams is shown in the following equation:

$$R_i = \frac{\theta_{cy,A} + \theta_{cy,B}}{\theta_{by,L} \left( 1 + \frac{h_i}{L_{cb,L}} \right) + \theta_{by,R} \left( 1 + \frac{h_i}{L_{cb,R}} \right)}$$  \hspace{1cm} (4.7)

Substituting the yield rotation equations and re-writing the formula:

$$R_i = 1.25 \left[ \frac{h_{x,A} + h_{x,B}}{h_{v,A} + h_{v,B}} \right] \frac{L_{b,L} L_{cb,L}}{h_{b,L} \left( L_{cb,L} + h_c \right)} + \frac{L_{b,R} L_{cb,R}}{h_{b,R} \left( L_{cb,R} + h_c \right)}$$  \hspace{1cm} (4.8)

The parameters of Equation (4.8) are shown in Figure (4.9). The ratio of the yield rotation capacities is calculated for each joint separately. The deformation based sway index is introduced as the average of the ratios $(R_i)$ for all joints at each floor level as shown in the following equation:

$$S_{ld} = \frac{\sum_{i=1}^{n} R_i}{n}$$  \hspace{1cm} (4.9)

Figure 4.9: Geometric parameters for the deformation based sway index
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The expected logic behind the resulting values of the index is that if the beams are more flexible than the columns, which is reflected by the fact that the values of yield rotations of the beams are higher than the columns, a beam sway mechanism is expected to form. On the other hand, if the columns are more flexible than the beams in terms of yield rotations, a column sway mechanism is expected to form. The limits of the deformation based index will be discussed in detail from the predicted side-sway mechanism of individual case study frames using numerical modelling.

The previous logic can be explained from the results of individual section analysis of each case study frame. For example, as shown in Figure (4.10), taking the case study-1 which is designed only for gravity load, it is expected to have a column sway mechanism. The plot shows the idea of having larger yield rotation of columns than beams, which means that columns are more flexible. Another selected example for a frame exhibiting a beam sway mechanism, case study-6, with strong columns in both stiffness and strength as shown in Figure (4.15). The beams in this case are more flexible than the columns and weaker as reflected by larger yield rotation. The plots explaining the logic of using the new deformation based index for the case studies are shown in Figure (4.10) to (4.16).

Figure 4.10 : Relative flexibility of beams and columns for case study-1

Figure 4.11 : Relative flexibility of beams and columns for case study-2
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Figure 4.12: Relative flexibility of beams and columns for case study-3

Figure 4.13: Relative flexibility of beams and columns for case study-4

Figure 4.14: Relative flexibility of beams and columns for case study-5
Figure 4.15: Relative flexibility of beams and columns for case study-6

Figure 4.16: Relative flexibility of beams and columns for case study-7
5 NONLINEAR STATIC PUSHOVER ANALYSIS OF CASE STUDIES – BARE FRAMES

In this chapter, nonlinear static pushover analyses of case study bare frames are performed using the nonlinear finite element analysis program SeismoStruct (SeismoSoft, 2008). The objective is to capture the most probable side-sway mechanism of the frames using advanced computational modelling which has been proven to give reliable estimate to the real performance of the structures under severe earthquake excitation (SeismoSoft, 2008). The innovative displacement based adaptive pushover analysis is used for this study. The assessment criteria are based on global response parameters.

5.1 Review of pushover analysis

Pushover analysis, also known as collapse analysis, is a nonlinear static monotonic lateral force–displacement analysis in which the mathematical model of the multi degree-of-freedom structure is subjected to a distribution of incrementally increasing lateral forces until the stability limit of the structure is reached. The pushover analysis can establish the capacity curve (pushover curve) of the structure, i.e. the path taken to reach the strength and ductility capacities of the structure, including the sequence of cracking, yielding and failure of components. A comprehensive review of the pushover methods has been carried out by Pinho et al. (2005) is summarized in the following paragraphs.

According to recently introduced code provisions, such as FEMA-356 (2000) and Eurocode 8 (CEN, 2003), pushover analysis should consist of subjecting the structure to an increasing vector of horizontal forces with an invariant pattern. Both the force distribution and target displacement are based on the assumptions that the response is controlled by the fundamental mode and the mode shape remains unchanged until collapse occurs. Two lateral load patterns, namely the first mode proportional and the uniform, are recommended to approximately bound the likely distribution of the inertia forces in the elastic and inelastic range, respectively.

A number of recent studies, well summarized in the FEMA-440 (ATC, 2005) report, raise significant doubts on the effectiveness of these conventional force-based pushover methods in estimating the seismic demand throughout the full deformation range: (i) inaccurate prediction
of deformations when higher modes are important and/or the structure is highly pushed into its nonlinear post-yield range, (ii) inaccurate prediction of local damage concentrations, responsible for changing the modal response, (iii) inability of reproducing peculiar dynamic effects, neglecting sources of energy dissipation such as kinetic energy, viscous damping, and duration effects, (iv) difficulty in incorporating three-dimensional and cyclic earthquake loading effects. Krawinkler and Seneviratna (1998) summarised the above with a single statement; fixed load patterns in pushover analysis are limiting, be they first modal or multimodal derived, because no fixed distribution is capable of representing the dynamic response throughout the full deformation range.

The aforementioned problems may constitute the reason for the unsatisfactory estimates of the seismic demands by current force-based pushover methods, mainly in irregular structures, due to the fact that the force control does not necessarily represent the actual dynamic response, as the distribution of forces in a dynamic analysis, even with a dominant first mode, do not exhibit a constant multiplier when inelastic mechanisms are activated. On the other hand, it is commonly agreed that the application of a constant displacement profile would force a predetermined and possibly inappropriate response mode, concealing important structural characteristics and concentrated inelastic mechanisms at a given location.

As a result, recent years have witnessed the introduction of the so-called adaptive pushover methods, which feature the ability to account for the effect that higher modes of vibration and progressive stiffness degradation might have on the distribution of seismic forces: the latter near collapse may differ significantly to that at the start of the analysis. The additional modelling and computational effort, with respect to conventional pushover procedures, is negligible.

The recent development of adaptive procedures has led to an improvement of the agreement between static and dynamic analysis, thanks to the consideration of: (i) spectrum scaling, (ii) higher modes contributions, (iii) alteration of local resistance and modal characteristics induced by the accumulated damage, (iv) load updating according to the eigen-solutions from an instantaneous nonlinear stiffness and mass matrix.

An adaptive load profile has the advantage of loading the structure according to its current dynamic characteristics at each step of the analysis, but despite its conceptual superiority, current force-based adaptive pushover features a relatively minor advantage over its traditional non-adaptive counterpart, particularly in what concerns the estimation of deformation patterns of buildings, which are poorly predicted by both types of analysis. The main reason for such underperformance seems to be the quadratic modal combination rules (SRSS, CQC) used in computing the adaptive updating of the load vector: currently implemented rules will inevitably lead to monotonically increasing load vectors, since the possibility of sign change in applied loads at any location is precluded, whilst it may be needed to represent the uneven redistribution of forces after an inelastic mechanism is triggered at some location.
5.2 Introduction to the displacement based adaptive pushover analysis (DAP)

The innovative displacement-based adaptive pushover procedure (DAP) is proposed by Antoniou and Pinho (2004) to overcome the difficulties in the force based adaptive pushover analysis. The employment of a displacement loading profile, entirely feasible within an adaptive pushover framework, since a displacement-based algorithm allows the possibility of introducing the reversal of at least the shear distributions, as a result of the structural equilibrium to the applied displacement pattern.

In this manner, not only the pushover analysis leads to more accurate results (deformation profiles and capacity curves) but it also becomes coherent with recent seismic design/assessment trends where the direct use of displacements, as opposed to forces, is preferred as recognition of the conspicuous evidence that seismic structural damage is in fact induced by response deformations.

The innovative displacement-based adaptive pushover procedure (DAP) proposed is assessed through an extensive comparative study involving different pushover methods, either single or multi mode, adaptive or conventional, and dynamic nonlinear analysis of reinforced concrete buildings and bridges subjected to diverse acceleration records. The deemed “true” dynamic response has been determined with the envelopes derived from the recent Incremental Dynamic Analysis procedure (Vamvatsikos and Cornell, 2002), whereby a structure is subjected to a series of nonlinear time-history analysis of increasing intensity. In case of buildings, the new approach manages to provide much improved response predictions, throughout the entire deformation range, in comparison to those obtained by force-based methods: prediction of the global behaviour, as well as of the deformed shape, proved to be very effective.

It is important to observe that a static procedure will never be able to completely replace a dynamic analysis; nevertheless, a methodology is searched to obtain response information reasonably close to that predicted by the nonlinear time history analyses. The innovative displacement-based adaptive pushover method is therefore shown to constitute an extremely appealing displacement-based tool for structural assessment, fully in line with the recently introduced deformation- and performance-oriented trends in the field of earthquake engineering: simplified performance-based assessment and design procedures can be defined, where the use of adaptive pushover is explicitly or implicitly included.
5.3 Displacement based adaptive pushover analysis parameters

Being an advanced static analysis method, adaptive pushover requires the definition of a number of additional parameters.

5.3.1 Loading phases

The Adaptive Load Control solution procedures are used. In pushover analysis, the applied loading usually consists of permanent gravity loads in the vertical (z) direction and incremental loads in one or both transversal (x & y) directions. The magnitude of increment load which in this case displacement $U_i$ at any given analysis step $i$ is given by the product of its nominal value $U_o$, defined by the user in the Applied Loading module, and the load factor $\lambda$ at that step:

$$U_i = \lambda U_o$$  \hspace{1cm} (5.1)

5.3.2 Type of scaling

The normalised modal scaling vector, used to determine the shape of the load vector (or load increment vector) at each step is obtained using (Displacement-based Scaling) in which scaling vector reflects the modal displacement distribution at that step.

The normalized modal scaling vector $D$, used to determine the shape of the load vector (or load increment vector) at each step, is computed at the start of each load increment. In order for such scaling vector to reflect the actual stiffness state of the structure, as obtained at the end of the previous load increment, an eigenvalue analysis is carried out. Modal loads can be combined by using either the Square Root of the Sum of the Squares (SRSS) or the Complete Quadratic Combination (CQC) method.

Displacement-based scaling refers to the case whereby storey displacement patterns $D_i$ are obtained directly from the eigenvalue vectors, as described in Equation (5.2), where $i$ is the storey number and $j$ is the mode number, $\Gamma_j$ is the modal participation factor of the $j$th mode, $\Phi_{ij}$ is the mass normalized mode shape value for the $i$th storey and the $j$th mode, and $n$ stands for the total number of modes.

$$D_i = \sqrt{\sum_{j=1}^{n} D_{ij}^2} = \sqrt{\sum_{j=1}^{n} (\Gamma_j \Phi_{ij})^2}$$  \hspace{1cm} (5.2)
Since only the relative values of storey displacements \(D_i\) are of interest in the determination of the normalised modal scaling vector \(D\), which defines the shape, not the magnitude, of the load or load increment vector, the displacements obtained by Equation (5.2) are normalised so that the maximum displacement remains proportional to the load factor, as required within a load control framework as described in Equation (5.3).

\[
\bar{D}_i = \frac{D_i}{\max D_i}
\]  

(5.3)

### 5.3.3 Type of updating

Once the normalised scaling vector, \(\bar{D}_t\), and load factor \(\Gamma_t\) or load factor increment \(\Delta\Gamma_t\) have been determined, and knowing also the value of the initial nominal load vector \(U_0\), the loading vector \(U_t\) at a given analysis step \(t\) can be updated using the incremental updating, in which the load vector for the current step is obtained by adding to the load vector of the previous step (existing balanced loads) as shown in Figure (5.1), a newly derived load vector increment, computed as the product between the current load factor increment, the current modal scaling vector and the initial user-defined nominal load vector as give in Equation (5.4).

\[
U_t = U_{t-1} + \Delta\lambda_t \bar{D}_i U_0
\]  

(5.4)

![Figure 5.1: Updating of the loading displacement vector.](image)

### 5.3.4 Spectral amplification

Previous research (Mwafy and Elnashai, 2001; Antoniou, 2002) indicated that considering the spectral amplification of each particular mode in the computation of the \(D_{ij}\) could contribute to an improvement in the similitude between static pushover and dynamic inelastic analysis results. A single constant displacement response spectrum according to EC8 specifications shown in Figure (5.2) derived for an equivalent viscous damping value of 5\% was used throughout each analysis.
The modal interstorey drifts are weighted by the spectral displacement $S_d$ value at the instantaneous period of that mode as described in Equation (5.5), so as to take into account the effects that the frequency content of a particular input time-history or spectrum have in the response of the structure being analysed.

$$\Delta_i = \sqrt{\sum_{j=1}^{n} \Delta_{i,j}^2} = \sqrt{\sum_{j=1}^{n} \left[ \Gamma_j \left( \phi_{j,1} - \phi_{j,1-j} \right) S_{d,j} \right]^2}$$  \hspace{1cm} (5.5)

Equations 4.6 to 4.9 (CEN, 2003) are the expressions that define the horizontal design spectrum, and then Table 4.1 specifies the values used for this study.

$$0 \leq T \leq T_B : S_d(T) = a_g \cdot S \cdot \left[ \frac{2}{3} + \frac{T}{T_B} \left( \frac{2.5}{q} - \frac{2}{3} \right) \right]$$  \hspace{1cm} (5.6)

$$T_B \leq T \leq T_C : S_d(T) = a_g \cdot S \cdot \frac{2.5}{q}$$  \hspace{1cm} (5.7)

$$T_C \leq T \leq T_D : S_d(T) = a_g \cdot S \cdot \frac{2.5}{q} \left[ \frac{T_C}{T} \right] \geq \beta \cdot a_g$$  \hspace{1cm} (5.8)

$$T_D \leq T : S_d(T) = a_g \cdot S \cdot \frac{2.5}{q} \left[ \frac{T_C T_D}{T^2} \right] \geq \beta \cdot a_g$$  \hspace{1cm} (5.9)

Table 5.1: Values of the parameters describing the design spectra (CEN, 2003)

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<tr>
<th>Parameter</th>
<th>Value</th>
<th>Observation</th>
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<td>$S_d(T)$</td>
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<td>Design spectrum</td>
</tr>
<tr>
<td>$T$</td>
<td></td>
<td>Vibration period of single-degree-of-freedom system</td>
</tr>
<tr>
<td>$a_g$</td>
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<td>Design ground acceleration on type B ground</td>
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<tr>
<td>$T_B$</td>
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<td>Lower limit of the period of the constant spectral acceleration branch</td>
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<td>$T_C$</td>
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<td>Upper limit of the period of the constant spectral acceleration branch</td>
</tr>
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<td>$T_D$</td>
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<td>Value defining the beginning of the constant displacement response range spectrum</td>
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<tr>
<td>$\beta$</td>
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<td>Lower bound for the horizontal design spectrum</td>
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Figure 5.2: Design displacement spectrum according to EC8 for soil type B-Seismic Zone 1

5.4 Member level performance criteria and damage assessment

Within the context of performance-based engineering, it is important to identify the instants at which different performance limit states (e.g., non-structural damage, structural damage, and collapse) are reached. This can be efficiently carried out in SeismoStruct through the definition of performance criteria, whereby the attainment of a given threshold value of material strain, section curvature, element chord-rotation and/or element shear during the analysis of a structure is automatically monitored by the program. Within the context of a fibre-based modelling approach, such as that implemented in SeismoStruct, material strains do usually constitute the best parameter for identification of the performance state of a given structure.

Two performance levels are defined for this study. The first performance level is related to moderate structural damage band as described in Displacement based earthquake loss assessment (DBELA) methodology (Crowley et al., 2004) in which member flexural strengths could be achieved, and some limited ductility developed, provided that concrete spalling in plastic hinges did not occur. For assessment of reinforced concrete buildings, limit values of $\varepsilon_c = 0.0035-0.004$ and $\varepsilon_s = 0.01-0.015$ are suggested. Note that similar values are normally considered appropriate for ultimate limit state when designing for gravity loads. The validity of these strain limits can be determined as follows. Typically, spalling of concrete is initiated at extreme fibre compression strains between $\varepsilon_c = 0.006$ and 0.01 (Calvi, 1999). Thus, the limit of 0.004 is a conservative estimate of the onset of structural damage. The strain limit of $\varepsilon_s = 0.015$ was determined to ensure that residual crack widths would not exceed 1.0 mm.

The second performance level considered is related to extensive structural damage band in which significant repair required to building, wide flexural or shear cracks, buckling of
longitudinal reinforcement may occur. This may correspond to local deformations in the critical section in the order of $\varepsilon_c = 0.006-0.01$ and $\varepsilon_s = 0.03-0.04$. It may be worth noticing that this limit state corresponds essentially to what is normally defined as an ultimate, or collapse limit state in most codes of practice. Summary of the considered performance limit states are listed in Table 5.2.

**Table 5.2 : Performance criteria for structural members.**

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<th>Performance Level</th>
<th>Material strain limit states</th>
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</thead>
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</tr>
<tr>
<td>Moderate – Member post- yield limit state</td>
<td>0.0035 – 0.004</td>
</tr>
<tr>
<td>Extensive – Member Collapse limit state</td>
<td>0.006 – 0.01</td>
</tr>
</tbody>
</table>

### 5.5 Global structural level damage assessment

For global structural level, the monitored quantity is the base shear ($V$) versus top displacement ($d$) as shown in Figure (5.3). Horizontal forces ($V_i$) of the support nodes were added and plotted against the horizontal displacement of the top floor.

![Figure 5.3 : Base shear versus global drift monitoring (after Papanikolaou et al. 2005)](image)

### 5.5.1 Global yielding criteria

Since the yield point is not clear in the plot of base shear versus top displacement, an idealized elasto-plastic system was assumed to find the approximate yield point in the global response of the structure. Following the procedures of EC8 (CEN 2003), the yield force $F_y^*$, which represents also the ultimate strength of the idealized system, is equal to the base shear force at the formation of the plastic mechanism. The initial stiffness of the idealized system is
determined in such a way that the areas under the actual and the idealized force – deformation curves are equal as shown in Figure (5.4).

Based on this assumption, the yield displacement of the idealised system $d_y^*$ is given by:

$$
    d_y^* = 2 \left( d_m^* - \frac{E_m^*}{F_y^*} \right) 
$$

(4.10)

Where $E_m^*$ is the actual deformation energy up to the formation of the plastic mechanism.

![Figure 5.4: Idealized elasto-plastic force displacement relationship according. (EC8,CEN 2003).](image)

5.5.2 Global collapse criteria

Significant strength and stiffness degradation can be a criterion of the collapse points on the pushover force displacement curves. According to the Turkish earthquake code (2007), a reliable measure of structural collapse can be defined when there is 30% loss of shear resistance at any storey level. This can be measured using the member collapse limit state by neglecting the columns that reached this limit state at its ends in resisting lateral loads until the load increment at which the limit of 30% reduction of shear resistance is reached. This loading step is considered as the collapse point.

5.5.3 Inter-storey drift monitoring

Lateral drifts are the main cause of structural damage in buildings subjected to earthquake ground motions. Additionally, lateral drifts are also responsible for earthquake-induced damage to many types of non-structural elements in buildings. Based on these observations, several recent studies have shown that present criteria for the seismic design of new structures and for the seismic evaluation of existing structures can be improved if they are based on the explicit consideration of lateral deformation demands as the main seismic design parameter rather than lateral forces (Jeong and Elnashai 2004).
Jeong and Elnashai (2004) stated that during the preliminary design of new buildings or for a rapid seismic evaluation of existing buildings there is a need for estimating the maximum lateral displacements that can occur in the building. On the global structural level, the inter-storey drift (ID) is one of the simplest and most commonly used damage indicators. For both global yielding as well as global collapse limits defined for each pushover capacity curve, the inter-storey drift (ID) profile is obtained.

It should be noted that several ID values corresponding to collapse for a building have been suggested by different researchers. At values in excess of the collapse limit, it is assumed that significant P-Δ effect leads to failure of a building. An ID of 2.5% has been suggested by SEAOC (1995) as the collapse limit for three-quarters of RC buildings as shown in Table (5.3).

**Table 5.3 : Performance levels and damage descriptions classified according to the ID ratio, (SEAOC – Vision 2000, 1995)**

<table>
<thead>
<tr>
<th>Performance level</th>
<th>Overall building damage</th>
<th>Transient drift</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fully operational</td>
<td>Negligible</td>
<td>ID &lt; 0.2%</td>
</tr>
<tr>
<td>Operational</td>
<td>Light</td>
<td>0.2% &lt; ID &lt; 0.5%</td>
</tr>
<tr>
<td>Life Safety</td>
<td>Moderate</td>
<td>0.5% &lt; ID &lt; 1.5%</td>
</tr>
<tr>
<td>Near collapse</td>
<td>Severe</td>
<td>1.5% &lt; ID &lt; 2.5%</td>
</tr>
<tr>
<td>Collapse</td>
<td>Complete</td>
<td>2.5% &lt; ID</td>
</tr>
</tbody>
</table>

5.6 **DAP analyses results for the case study bare frames**

In the following section, the displacement based adaptive pushover analyses results for the case study bare frames are discussed in terms of global structural damage criteria mentioned in the previous section.

The pushover capacity curve as well as the elasto-plastic bilinear approximation are presented in the following for each case study relating the development of lateral shear resistance with increasing displacement demand on the frame.

The vertical distribution of displacement over structure height is also presented for each case study in terms of the displacement profile, inter-storey drift and the normalized displacement profile as a function of the displacement of the floor with the highest inter-storey drift. The displacement and drift profiles have been captured at the global structural yield as well as the collapse step. The target is to observe the difference in the side-sway mechanism at the structural yield and collapse steps.
5.6.1 DAP results for case study-1

The Case study-1 represents a reinforced concrete frame designed only for gravity load without seismic design provisions (see Section 3.1). The capacity curve is shown in Figure (5.5). The expected performance of this type of non-ductile frame is a soft storey mechanism (see Figure 5.6) due to the presence of strong-stiff beams and weak-flexible columns in terms of both strength and stiffness.

![Figure 5.5: Capacity curve for case study-1](image)

![Figure 5.6: Vertical distribution of lateral displacement for case study-1](image)
5.6.2 DAP results for case study-2

The Case study-2 represents a reinforced concrete frame designed only for gravity load without seismic design provisions (see Section 3.2). The capacity curve is shown in Figure (5.7). The expected performance of this type of non-ductile frames is a soft storey mechanism (see Figure 5.8) due to the presence of strong-stiff beams and weak-flexible columns in terms of both strength and stiffness. Storey mechanisms are developed at the second and the fourth storeys as shown in Figure (5.8) due to the sudden reduction of strength and stiffness at these storeys.

![Capacity Curve for Case Study-2](image)

**Figure 5.7 : Capacity curve for case study-2**

![Vertical Distribution of Lateral Displacement](image)

**Figure 5.8 : Vertical distribution of lateral displacement for case study-2**
5.6.3 DAP results for case study-3

The case study-3 is a four-storey bare frame known to have a soft storey at the third floor and which as can be seen from Figure (5.10) has been correctly identified with the DAP pushover analysis (see section 3.3). The latter is attributed to a drastic change in strength and stiffness at this level through a reduction in both the reinforcement content and the section dimensions in the columns between the second and third storeys. The capacity curve is shown in Figure (5.9).

![Capacity curve for case study-3](image1)

**Figure 5.9 : Capacity curve for case study-3**

![Vertical distribution of lateral displacement](image2)

**Figure 5.10 : Vertical distribution of lateral displacement for case study-3**
5.6.4 DAP results for case study-4

The case study-4 is a five storey frame following the Turkish practice of gravity load designed frames without seismic provisions (see section 3.4). The capacity curve is shown in Figure (5.11). The formation of a soft first storey is predicted according to the presence the reduced stiffness at that storey as can be seen in Figure (5.12).

![Capacity curve for case study-4](image1)

**Figure 5.11 : Capacity curve for case study-4**

![Vertical distribution of lateral displacement for case study-4](image2)

**Figure 5.12 : Vertical distribution of lateral displacement for case study-4**
5.6.5 DAP results for case study-5

The case study-5 is a seven storey frame following the Turkish practice of moment resisting frames with embedded wide beams frames with seismic provisions (see section 3.5). The capacity curve is shown in Figure (5.13). The beam hinging pattern is observed according to the design of the frame with flexible beams as shown in Figure (5.14).

![Capacity curve for case study-5](image)

**Figure 5.13 : Capacity curve for case study-5**

![Vertical distribution of lateral displacement](image)

**Figure 5.14 : Vertical distribution of lateral displacement for case study-5**
5.6.6 DAP results for case study-6

The case study-6 is a seven storey frame following the seismic design provisions of the Yugoslavian code 1984 of moment resisting frames (see section 3.6). The capacity curve is shown in Figure (5.15). The predicted mechanism is a beam hinging pattern according to the design of the frame with capacity design principle of strong columns and week beams as shown in Figure (5.16).

![Capacity Curve for Case Study-6](image)

**Figure 5.15 : Capacity curve for case study-6**

![Vertical Distribution of Lateral Displacement](image)

**Figure 5.16 : Vertical distribution of lateral displacement for case study-6**
5.6.7 DAP results for case study-7

The case study-7 is a seven storey frame following the seismic design provisions of the Portuguese code of moment resisting frames (see section 3.7). The capacity curve is shown in Figure (5.17). The predicted mechanism is a beam hinging pattern according to the design of the frame with capacity design principle of strong columns and week beams as shown in Figure (5.18).

![Figure 5.17: Capacity curve for case study-7](image1)

![Figure 5.18: Vertical distribution of lateral displacement for case study-7](image2)
6 NONLINEAR STATIC PUSHOVER ANALYSIS OF CASE STUDIES – MASONRY INFILLED RC FRAMES

6.1 Introduction

Unreinforced masonry infills are normally treated as non-structural elements. However, unlike most non-structural components, they can develop a strong interaction with the bounding frames when subject to earthquake loads and, therefore, contribute significantly to the lateral stiffness and load resistance of the structure. In spite of research efforts that have spanned several decades, the performance of these structures in a severe earthquake remains a major controversy among structural engineers and researchers today.

There is a need for a systematic study for the assessment and quantification of the effect of masonry infill panels on the seismic performance and possible seismic damage of practical masonry infilled RC framed building structures under the influence of earthquake excitation. The displacement-based adaptive pushover analysis procedure described in Chapter 4 using the numerical modelling software SeismoStruct (SeismoSoft 2008) is applied to the case study frames including masonry infills. The objective is to observe the effect of masonry infill panels on the global side-sway collapse mechanism of the reinforced concrete frames and compare the results with the behaviour of bare frames. The proposed sway potential index discussed in chapter 3 is expected to be updated according to the results obtained in this chapter to take into account the participation of masonry infill panels in seismic performance for both cases of fully infilled frames and irregularly infilled with first open storey “Pilotis” frames.

6.2 Review of seismic performance of masonry infilled RC frames

Masonry infill panels, confined by reinforced concrete frames on all four sides, play a vital role in resisting the lateral seismic loads on buildings. It has been shown experimentally that masonry infill panels have a very high initial lateral stiffness and low deformability. Thus, introduction of infill panels in RC frames changes the lateral-load transfer mechanism of the structure from predominant frame action to predominant truss action (Murty and Jain 2000),
as shown in Figure (6.1), which is responsible for a reduction in bending moments and an increase in axial forces in the frame members.

![Figure 6.1: Change in lateral-load transfer mechanism due to masonry infills. (after Murty and Jain 2000).](image)

There is strong evidence that masonry infills enhance the lateral strength of framed building structures under severe earthquake loads and have been successfully used to strengthen the existing moment resisting frames in some countries. However, there is a common misconception that masonry infills in reinforced concrete can only enhance their lateral load performance and must therefore always be beneficial to the earthquake resistance of the structure. There are numerous cases of seismic damage that can be attributed to modification of the dynamic response parameters of the basic structural frame by so-called nonstructural masonry infills.

Irregular distribution of the infill panels and openings is a common practice. Often infill panels are rearranged to suit the changing functional needs of the occupants, the changes being carried out without considering their adverse effects on the overall structural behaviour because infill panels are generally regarded as nonstructural elements of buildings (Kaushik et al. 2006). Masonry infills panels can be distributed in RC frames in several patterns, for example, as shown in Figure (6.2).

![Figure 6.2: Different arrangements of masonry infill panels in RC frame (Kaushik et al. 2006).](image)
Chapter 6. Nonlinear Static Analysis of Case Studies – Masonry Infilled RC Frames

The addition of masonry infill panels to an originally bare moment resisting frame increases the lateral stiffness of the structure, thus shifting the natural time period on the earthquake response spectrum in the direction of higher seismic base and storey shears, and attracting earthquake forces to parts of structures not designed to resist them. Furthermore, if the structure is designed to act as a moment resisting frame with a ductile response to the design level earthquakes, neglecting the contribution of infills, the stiffening effect of the infills may increase the column shears resulting in the development of plastic hinges at the top of the columns that are in contact with the infill corners.

As discussed by El-Dakhakhni et al. (2003), ignoring the effect of the infill in stiffening and strengthening the surrounding frame is not always a conservative approach, since the stiffer the building, usually, the higher seismic loads it attracts. If the panel is overstressed and hence failed partially or wholly, the high forces previously attracted and carried by the stiff infilled frame will be suddenly transferred to the more flexible frame after the infill is partially or fully damaged. Because of the complexity of the problem and the absence of a realistic, yet simple, analytical model, the effect of masonry-infill panels is often neglected in the nonlinear analysis of building structures. Such an assumption may lead to substantial inaccuracy in predicting the lateral stiffness, strength, and ductility of the structure. It will also lead to uneconomical design of the frame since the strength and stiffness demand on the frame could be largely reduced.

Fardis (2000) carried out a European Commission sponsored study on the design of infilled RC structures. The study concluded that for the infills with regular distribution, the overall effect of infills is found to be beneficial and their main effect is on energy dissipation. Additionally, the only adverse effect for regularly distributed infilled structures is a tendency for drift concentration in the bottom storey and for structures with an open bottom storey the concentration of drift and structural damage of the columns become significant.

Murty and Jain (2000) carried out experimental testing to study the influence of masonry infills on seismic performance of RC frame buildings. The study found the influence of masonry infills to be beneficial. Infill panels increase strength, stiffness, overall ductility and energy dissipation of the building. Further, they dramatically decrease the deformation and ductility demand on RC frame members. These buildings therefore perform well in moderate earthquakes. Detrimental effects of infills, such as short column effect, soft-storey effect, and torsion, are however a concern.

One of the lessons learnt from past experiences in earthquakes is that abrupt changes in stiffness along the height of a building due to irregular distribution of masonry infill panels over the elevation of the building frame can unfavourably and sometimes catastrophically affect the seismic performance of the frame. The complexity in predicting the seismic performance of masonry infilled frames with irregular distribution of masonry infill panels reveals the importance of modelling and analyzing the structural contribution of the masonry infill panels to the seismic response of the infilled frame.
Chapter 6. Nonlinear Static Analysis of Case Studies – Masonry Infilled RC Frames

Understanding and assessing the seismic performance of masonry-infilled RC frames presents a difficult problem in structural engineering. Analytical tools to evaluate the complicated frame-infill interaction and the resulting failure mechanisms need to be built on the fundamental principles of mechanics and sound engineering judgment.

6.3 Failure modes of masonry infilled RC frames

Different failure modes of masonry-infilled frames can be categorized into five distinct modes as summarized by El-Dakhakhni et al. (2003) based on the knowledge gained from both analytical and experimental studies during the last five decades:

1. The corner crushing (CC) mode represents crushing of the infill in at least one of its loaded corners, as shown in Figure (6.3a). This mode is usually associated with infill of weak masonry blocks surrounded by a frame with weak joints and strong members.

2. The sliding shear (SS) mode represents horizontal sliding shear failure through bed joints of a masonry infill, as shown in Figure (6.3b). This mode is associated with infill of weak mortar joints and a strong frame.

3. The diagonal compression (DC) mode represents crushing of the infill within its central region, as shown in Figure (6.3c). This mode is associated with a relatively slender infill, where failure results from out-of-plane buckling instability of the infill.

4. The diagonal cracking (DK) mode is seen in the form of a crack connecting the two loaded corners, as shown in Figure (6.3d). This mode is associated with a weak frame or a frame with weak joints and strong members infilled with a rather strong infill.

5. The frame failure (FF) mode is seen in the form of plastic hinges in the columns or the beam–column connection, as shown in Figure (6.3e). This mode is also associated with a weak frame or a frame with weak joints and strong members infilled with a rather strong infill.
El-Dakhakhni et al. (2003) noted that only the first two modes, the (CC) and the (SS) modes, are of practical importance, since the third mode (DC) occurs very rarely and requires a high slenderness ratio of the infill to result in out of-plane buckling of the infill under in-plane loading. This is hardly the case when practical panel dimensions are used, and the panel thickness is designed to satisfy the acoustic isolation and fire protection requirements. The fourth mode (DK) should not be considered a failure mode, due to the fact that the infill can still carry more loads after it cracks. The fifth mode (FF) is of importance in the case of reinforced concrete frames.

### 6.4 Review of current guidelines for seismic evaluation of masonry infilled RC frames

A comprehensive review of the current guidelines for seismic evaluation of masonry infilled RC frames has been carried out by Bell and Davidson (2001) which is summarized in the following section.

#### 6.4.1 New Zealand procedures

Section 6.3 of the NZNSEE draft Earthquake Risk Building Document “The assessment and improvement of the structural performance of earthquake risk buildings” (NZNSEE, 2002) addresses moment resisting frame elements with masonry infill panels. The main thrust of the document is that if the infill panels have a significant effect on the seismic response and are expected to suffer damage without collapse, then it is likely that a soft storey mechanism will form. Diagonal failure modes can be expected to degenerate into a sliding shear failure mode, leading to hinging of columns between floors. This is illustrated in Figure (6.4). The document states that for exterior columns, hinges form top and bottom and at mid-height,
while for interior columns, hinges form close to quarter points. Ductility capacity and demand may be assessed as for a column sway mechanism with the reduced column height. It should be emphasized that the presence of infills modifies and magnifies the shear demands on the frame members by shortening the distance between in-span plastic hinges.

![Modelling the adverse effect of an infill panel on the performance of the perimeter frame](image)

**Figure 6.4**: Modelling the adverse effect of an infill panel on the performance of the perimeter frame (NZNSEE, 2002).

The recommendation of (Paulay and Priestley, 1992) is that masonry infill panels be modelled as equivalent concentric diagonal struts, based on an effective width of 0.25 times the diagonal length as a conservative estimate. This is illustrated in Figure (6.5). It is recommended that where sliding or diagonal compression failure may occur, frames should be designed elastically due to the concentration of deformation in the first storey.

![Deformation under shear load](image)

**Figure 6.5**: Deformation under shear load (Paulay and Priestley, 1992).

(Crisafulli et al., 2000) provided a review of macro and micro models, and reported on some comparative studies between equivalent strut models and finite element models. Amongst the conclusions were that single strut models can provide an adequate estimation of stiffness of the infilled frame, however multi-strut models are required to obtain realistic values of the bending moments and shear forces in the frames.
6.4.2 United States procedures

The NEHRP Guidelines for the seismic rehabilitation of buildings (FEMA-273, 1997) is an extensive document for use in the design and analysis of seismic rehabilitation projects. FEMA-273, section 7.5 addresses masonry infilled systems. It specifies that masonry infill panels shall be represented as equivalent diagonal struts. The struts may be placed concentrically across the diagonals, or eccentrically to directly evaluate the infill effects on the columns. This is illustrated in Figure (6.6).

![Figure 6.6: Compression Strut Analogy – Eccentric and Concentric Struts (FEMA-273 1997).](image)

The shear behaviour of masonry infill panels is considered as a deformation controlled action. FEMA-273 provides deformation acceptance criteria. The linear procedure involves comparing the design elastic shear force in a panel with the factored expected shear strength of the panel. Panel strength is given by the shear sliding strength with no enhancement for axial stress. FEMA-273 specifies strength requirements for column members adjacent to infill panels. Column actions may be evaluated through the application of the horizontal component of the expected infill strut force at a specified distance from the columns ends. Shear force demand may however be limited by the moment capacities of the column of reduced length. The reinforced concrete column shear failure mode typically occurs near the frame joints and is associated with stiff and/or strong infills.

Deformation capacity guidelines recommended by FEMA-306 (1998) are given in the form of interstorey drift ratios. These vary from 1.5% for brick masonry to 2.5% for ungrouted concrete block masonry. As diagonal cracking is initiated at drifts of 0.25% and essentially complete by about 0.5% this represents a high level of ductility in the panel system.

6.4.3 European procedures

Eurocode 8 (CEN 2003) contains provisions for the design of infilled RC frames (section 4.3.6) as additional measures for the non-engineered masonry infills that in contact with the frame without structural connections and that are considered in principle as non-structural elements. The irregularities due to masonry infills in plan and elevation are discussed. EC8
recommended that if there are considerable irregularities in elevation such as a drastic reduction of infills in one or more storeys compared to the others, the seismic action effects in the vertical elements of the respective storeys shall be increased by a magnification factor. Section (5.9) in part-1 provided special measures for the local effects of the infill panels on the critical column length, confinement requirements and shear force transmitted to the columns. In the EC8 though there are recommended that masonry infills should be taken into account in the analysis of the frame though the modelling of the masonry infill panels is not specified.

6.5 Description of the infill panel model

A four-node masonry panel element, developed and initially programmed by Crisafulli (1997) and implemented in SeismoStruct by Blandon (2005), was used for the modelling of the nonlinear response of infill panels in framed structures. Each panel is represented by six strut members; each diagonal direction features two parallel struts to carry axial loads across two opposite diagonal corners and a third one to carry the shear from the top to the bottom of the panel. This latter strut only acts across the diagonal that is on compression; hence its activation depends on the deformation of the panel. The axial load struts use the masonry strut hysteresis model, while the shear strut uses a dedicated bilinear hysteresis rule.

The modeling of the infill frames with the implemented hysteretic models requires the definition of a significant number of parameters, justifying this fact by the need of generality of the model. This approach allows different levels of detailing of the model, depending on the available information. However, in the common practice, this information is not available and some parameters have to be assumed or approximated based on existing references.

6.5.1 Masonry diagonal strut hysteresis model

The model consists of two compression struts in each diagonal of the panel as shown in Figure (6.7). The element has 4 external nodes, 4 internal nodes (connected to the frame corner) and 4 dummy nodes (located at a distance $h_z$ from the frame corner). The formulation to perform this process is based on the work of Crisafulli (1997). The dummy nodes are used to represent the local effect between the infill and the frame, and are placed at a distance equal to $h_z/3$. The internal nodes represent the frame-infill contact at the exterior part of the beam and column, and the internal forces are then transformed to the 4 exterior nodes where the element is connected to the frame.

The strut model is composed by two parts which are the hysteretic material model shown in Figure (6.8) and an area reduction rule shown in Figure (6.9) which describes the infill strength and stiffness degradation with cracking. For this endeavour the default values of SeismoStruct were used, which have been calibrated by Smyrou et al. (2006). The geometry and the material properties of the diagonal struts have been defined using the values and formulae reported in Table 6.1. Users are strongly advised to consult the publications of Crisafulli et al. (2000) and Smyrou et al. (2006) for further details on this model.
Chapter 6. Nonlinear Static Analysis of Case Studies – Masonry Infilled RC Frames

Figure 6.7: Infill panel element description implemented in SeismoStruct (Blandon-Uribe, 2005)

Figure 6.8: Typical cyclic response with small cycle hysteresis (Crisafulli, 1997)

Figure 6.9: Variation of the area of the masonry strut as function of axial strain.
Table 6.1: Values and formulae used to define the properties of the diagonal masonry struts in SeismoStruct (adapted from Crowley and Pinho, 2006)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value / formula</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean diagonal Compressive strength, ( f'_{m\theta} )</td>
<td>2.5 MPa</td>
<td>Calvi et al. [2004]</td>
</tr>
<tr>
<td>Elastic Young’s Modulus, ( E_{m\theta} )</td>
<td>( 1000E_{m\theta} )</td>
<td>Crisafulli [1997]</td>
</tr>
<tr>
<td>Thickness of the infill panel, ( t_w )</td>
<td>0.25 m</td>
<td>Calvi et al. [2004]</td>
</tr>
<tr>
<td>Length of strut, ( d_w )</td>
<td>Calculated from geometry of the frame</td>
<td></td>
</tr>
<tr>
<td>Relative stiffness parameter, ( \lambda )</td>
<td>[ \lambda = \frac{E_{m\theta}t_w\sin(2\theta)}{4E_cI_p h_w} ]</td>
<td>Stafford Smith [1966]</td>
</tr>
<tr>
<td>Width of compressive strut, ( b_w )</td>
<td>( b_w = \frac{0.95h_w\cos\theta}{\sqrt{2h_w}} ) ( 0.1d_w \leq b_w \leq 0.4d_w )</td>
<td>Liauw and Kwan [1984]</td>
</tr>
<tr>
<td>Initial strut area, ( A_{m1} )</td>
<td>( A_{m1} = d_w b_w )</td>
<td>[Figure 6.9]</td>
</tr>
<tr>
<td>Residual strut area, ( A_{m2} )</td>
<td>( A_{m2} = A_{m1} \times (b_{w\text{cracked}}/b_{w\text{uncracked}}) )</td>
<td>[Figure 6.9]</td>
</tr>
<tr>
<td>Contact length, ( z )</td>
<td>( z = \frac{\pi}{2\lambda h_w} )</td>
<td>Stafford Smith[1966]</td>
</tr>
<tr>
<td>Horizontal offset, ( X_\theta )</td>
<td>Calculated from geometry of the frame</td>
<td></td>
</tr>
<tr>
<td>Vertical offset, ( Y_\theta )</td>
<td>Calculated from geometry of the frame</td>
<td></td>
</tr>
</tbody>
</table>

### 6.5.2 Shear strut hysteresis model

The adopted model shown in Figure (6.10) is capable of representing the shear behaviour when bond failure happens along the mortar joints. It is assumed that the behaviour of the latter is linear elastic while the shear strength is not reached. Unloading and reloading are also in the elastic range. Thus, the shear stress \( \tau \) is equal to the shear deformation \( \gamma \) times the shear modulus \( G_m \).

The shear strength results as the combination of two mechanisms, namely, bond strength and the friction resistance between the mortar joints and the bricks. The shear strength can thus be expressed as the sum of the initial shear bond strength \( \tau_o \) and the product of coefficient of friction \( \mu \) by the absolute value of the normal compressive force in the direction perpendicular...
to the bed joints. This approach to estimate shear resistance is pragmatically adopted by design codes, independently of the failure mechanism (shear friction failure, diagonal tension failure, compression failure) being developed in the infill panel. Users are strongly advised to consult the publications of Crisafulli et al. (2000), Blandon (2005) and Smyrou et al. (2006) for further details on this model. The parameters required to fully characterize the shear strut hysteresis model are listed in Table (6.2).

![Cyclic shear hysteresis model for shear strut element (SeismoStruct 2007).](image)

**Figure 6.10**: Cyclic shear hysteresis model for shear strut element (SeismoStruct 2007).

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>shear bond strength $\tau_o$</td>
<td>0.3 MPa</td>
</tr>
<tr>
<td>coefficient of friction $\mu$</td>
<td>0.7</td>
</tr>
<tr>
<td>Maximum shear strength $\tau_{\text{max}}$</td>
<td>1.0 MPa</td>
</tr>
<tr>
<td>Reduction shear factor $\alpha_s$</td>
<td>1.5</td>
</tr>
</tbody>
</table>

**Table 6.2**: Values and used to define the properties of the shear masonry struts in SeismoStruct (adapted from Smyrou et al. [2006])

### 6.6 DAP results of the case study masonry fully infilled frames

In the following section, the displacement based adaptive pushover analyses results for the fully infilled case-study frames are discussed in terms of global structural damage criteria mentioned in Chapter 5. The pushover capacity curve as well as the elasto-plastic bilinear approximation are presented for each case study relating the development of lateral shear resistance with increasing displacement demand on the frame.

The vertical distribution of displacement over structure height is presented in terms of displacement profile, inter-storey drift and the normalized displacement profile as a function
of the displacement of the floor with the highest inter-storey drift. The displacement profiles have been captured at the global structural yield as well as the collapse step.

6.6.1 DAP results of the gravity load designed frames group

Case studies 1 to 4 represent RC frames designed only for gravity load without seismic provisions. The pushover analyses have been carried out for these case studies after adding uniformly distributed masonry infill panels along the height of the frame. As will be shown in the following results, the predicted performance of this type of frames is a soft first storey mechanism. The fully infilled frames exhibit significantly higher peak and residual strength and initial stiffnesses than bare frames. It is observed that the masonry infills reduce the global lateral displacement of the frame.

6.6.1.1 DAP results for case study-1 fully infilled

The Case study-1 represents a reinforced concrete frame designed only for gravity load without seismic design provisions (see Section 3.1). The pushover capacity curve is shown in Figure (6.11) where it can be observed the large initial stiffness and peak strength compared to the bare frame. The infilled frame demonstrated a stiff load-displacement relationship followed by a sharp load decrease where About 40% of the peak strength was retained. The global yield step of the infilled frame occurred after reaching the peak strength at top drift 0.2% due to the sudden reduction of strength caused by the failure of the first storey infill panels, while in the case of the bare frame, the global yield step occurred before reaching the peak strength. The predicted behaviour of the frame is the formation of a first soft storey mechanism (see Figure 6.12) which is the same as the bare frame case (see section 5.6.1). The collapse step of the infilled frame occurred at a lower top drift (1.4%) than the bare frame (2.3%).
Chapter 6. Nonlinear Static Analysis of Case Studies – Masonry Infilled RC Frames

Figure 6.11: Capacity curve for case study-1 fully infilled

Figure 6.12: Vertical distribution of lateral displacement for case study-1 fully infilled

6.6.1.2 DAP results for case study-2 fully infilled

The Case study-2 represents a reinforced concrete frame designed only for gravity load without seismic design provisions (see Section 3.2). The pushover capacity curve is shown in Figure (6.13) where it can be observed the large initial stiffness and peak strength compared to the bare frame. The infilled frame demonstrated a stiff load-displacement relationship followed by a sharp load decrease where about 45% of the peak strength was retained. The
global yield step of the infilled frame occurred after reaching peak strength at a top drift 0.1% due to the sudden reduction of strength caused by the failure of the first storey infill panels, while in the case of the bare frame, the global yield step occurred before reaching the peak strength. The predicted behaviour of the frame is the formation of a first soft storey mechanism (see Figure 6.14), while in the case of bare frame, storey mechanisms developed at the second and the fourth storeys (see section 5.6.2). The collapse step of the infilled frame occurred at a top drift (0.54%) lower than the bare frame (5%).

Figure 6.13: Capacity curve for case study-2 fully infilled

Figure 6.14: Vertical distribution of lateral displacement for case study-2 fully infilled.
6.6.1.3 DAP results for case study-3 fully infilled

The case study-3 is a four-storey frame known to have a soft storey at the third floor due to the sudden reduction in strength and stiffness which has been identified in the case of the bare frame (see section 5.6.3). The pushover capacity curve is shown in Figure (6.15) where it can be observed the large initial stiffness and peak strength compared to the bare frame. The infilled frame demonstrated a stiff load-displacement relationship followed by a mild load decrease which does not follow the typical infilled frame behaviour. This can be explained by the presence of the strong column which has larger participation in resisting the lateral load that delayed the sudden failure of all infill panels at the first storey. About 50% of the peak strength was retained. The global yield step of the infilled frame occurred at nearly the peak strength at a top drift 0.4%. The predicted behaviour of the frame is the formation of a first soft storey mechanism (see Figure 6.16). The collapse step of the infilled frame occurred at a top drift (1.3%) lower than the bare frame (2.6%).

![Capacity Curve](image)

Figure 6.15 : Capacity curve for case study-3 fully infilled
6.6.1.4 DAP results for case study-4 fully infilled

The case study-4 is a five storey frame following the Turkish practice of gravity load designed frames without seismic provisions (see section 3.4). The capacity curve is shown in Figure (6.17). The infilled frame demonstrated a stiff load-displacement relationship followed by a sharp load decrease where about 50% of the peak strength was retained. The formation of a soft first storey is predicted according to the presence of a reduced stiffness at that storey as can be seen in Figure (6.18) which is the same behaviour as the case of the bare frame (see section 5.6.4). The global yield step of the infilled frame occurred after reaching the peak strength at a top drift 0.08%. The collapse step of the infilled frame occurred at nearly similar top drift (1.0%) as the bare frame (1.1%).

Figure 6.17: Capacity curve for case study-4 fully infilled
6.6.2 DAP results for case study-5 fully infilled (embedded beams frame)

The case study-5 is a seven storey frame following the Turkish practice of moment resisting frames with embedded wide beams frames with seismic provisions (see section 3.5). The pushover capacity curve is shown in Figure (6.19). The infilled frame demonstrated a stiff load-displacement relationship followed by a sudden reduction in strength due to the failure the infill panels at the second and the third storeys and about 85% of the peak strength was retained. At the collapse step, flexural hinging occurred at the bottom of the second storey columns and at the top of the fourth storey columns as can be seen in Figure (6.20) which is considered a partial storeys mechanism. The behaviour is different than the case of the bare frame where beam hinging pattern occurred (see section 5.6.5). The global yield step of the infilled frame occurred after reaching the peak strength at a top drift 0.4% compared to a value of 0.65% for the bare frame. The collapse step of the infilled frame occurred at a value of top drift (1.6%) lower than the bare frame (2.1%).

Figure 6.18: Vertical distribution of lateral displacement for case study-4 fully infilled
6.6.3 DAP results for the frames designed with seismic provisions group

Case Studies 6 and 7 represent frames following the seismic design provisions of moment resisting frames. Partial column sway mechanism was observed for both infilled frames. The typical behaviour of the infilled frames with high initial stiffness and peak strength was observed.
6.6.3.1 DAP results for case study-6 fully infilled

The case study-6 is a seven storey frame following the seismic design provisions of the Yugoslavian code 1984 of moment resisting frames (see section 3.6). The capacity curve is shown in Figure (6.21) where the infilled frame demonstrated a stiff load-displacement relationship followed by a sudden reduction in strength due to the failure the infill panels at the first three storeys and about 60% of the peak strength was retained. At the collapse step, flexural hinging occurred at the foundation level and at the top the third storey columns forming a partial mechanism up to the third floor level (see Figure 6.22). Beam hinging pattern was observed in the case of bare frame (see section 5.6.6). The global yield step of the infilled frame occurred after reaching the peak strength at a top drift 0.45% compared to a value of 0.64% for the bare frame. The collapse step of the infilled frame occurred at a top drift (1.6%) lower than the bare frame (3.2%).

![Capacity Curve](image)

Figure 6.21: Capacity curve for case study-6 fully infilled.
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6.6.3.2 DAP results for case study-7 fully infilled

The case study-7 is a four storey frame following the seismic design provisions of the Portuguese code of moment resisting frames (see section 3.7). The capacity curve is shown in Figure (6.23) where the infilled frame demonstrated a stiff load-displacement relationship followed by a sudden reduction in strength due to the failure of the infill panels at the first two storeys and about 60% of the peak strength was retained. At the collapse step, flexural hinging occurred at the foundation level and at the top of the second storey columns forming a partial mechanism up to the second floor level (see Figure 6.24). Beam hinging pattern was observed in the case of bare frame (see section 5.6.7). The global yield step of the infilled frame occurred after reaching the peak strength at a top drift 0.3% compared to a value of 0.83% for the bare frame. The collapse step of the infilled frame occurred at a top drift (2.0%) lower than the bare frame (3.3%).
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Figure 6.23: Capacity curve for case study-7 fully infilled

Figure 6.24: Vertical distribution of lateral displacement for case study-7 fully infilled
6.7 DAP results of the case studies masonry infilled Pilotis frames

A very widespread structural configuration in existing buildings is characterised by the absence of infill panels at the ground floor while they are present at the higher storeys. It is the so called "Pilotis" configuration characterised by a soft first storey. This configuration allows for a good use and distribution of the space at the ground floor but it is very dangerous from a seismic point of view, because the lateral response of these buildings is characterised by large rotation ductility demands concentrated at the extreme sections of the columns of the first storey, while the superstructure behaves like a rigid body. The presence of the masonry infilled panels increases the lateral stiffness, strength, and dissipation capacity of storeys above the first floor. This generally creates a structural vertical discontinuity of stiffness and strength.

As will be seen in the following results, The DAP prediction of all the case studies except the frame with embedded beams (case study-5) is the formation of a soft first storey, even with the frames with seismic design provisions. For the case study-5 that represents a frame with flexible embedded beams, a partial column sway mechanism is predicted by column hinging at the ground level and at the third floor level.

6.7.1 DAP results for case study-1 infilled Pilotis

The Case study-1 represents a reinforced concrete frame designed only for gravity load without seismic design provisions (see Section 3.1). The pushover capacity curve is shown in Figure (6.25) where it can be observed that the infilled pilotis frame demonstrated a typical load-displacement relationship similar to the bare frame. The global yield step of the infilled pilotis frame occurred before reaching the peak strength at top drift 0.15% which is lower than the bare frame case (0.3%). The predicted behaviour of the frame is the formation of a first soft storey mechanism (see Figure 6.26) which is the same as the bare frame case (see section 5.6.1) and the fully infilled frame (see section 6.6.1.1). The collapse step of the infilled pilotis frame occurred at a top drift (1.1%) lower than the fully infilled frame (1.4%) and the bare frame (2.3%).
Figure 6.25: Capacity curve for case study-1 infilled Pilotis

Figure 6.26: Vertical distribution of lateral displacement for case study-1 infilled Pilotis
6.7.2 DAP results for case study-2 infilled Pilotis

The Case study-2 represents a reinforced concrete frame designed only for gravity load without seismic design provisions (see Section 3.2). The pushover capacity curve is shown in Figure (6.27) which demonstrated a typical load-displacement relationship similar to the bare frame. The global yield step of the infilled pilotis frame occurred before reaching the peak strength at top drift 0.05% which is lower than the bare frame case (0.72%). The predicted behaviour of the frame is the formation of a first soft storey mechanism (see Figure 6.28) which is the same as the fully infilled frame (see section 6.6.1.2) while in the case of bare frame, storey mechanisms developed at the second and the fourth storeys (see section 5.6.2). The collapse step of the infilled pilotis frame occurred at a top drift (0.5%) which is very close to the case of fully infilled frame (0.54%) and lower than the bare frame (5%).

![Capacity Curve](image)

Figure 6.27 : Capacity curve for case study-2 infilled Pilotis
6.7.3 DAP results for case study-3 infilled Pilotis

The case study-3 is a four-storey frame known to have a soft storey at the third floor due to the sudden reduction in strength and stiffness which has been identified in the case of the bare frame (see section 5.6.3). The pushover capacity curve of the infilled pilotis frame is shown in Figure (6.29) which demonstrated a typical load-displacement relationship similar to the bare frame. The global yield step of the infilled pilotis frame occurred at top drift (0.1%) which is lower than the bare frame case (0.46%). The predicted behaviour of the frame is the formation of a first soft storey mechanism (see Figure 6.30) which is the same as the fully infilled frame (see section 6.6.1.3). The collapse step of the infilled pilotis frame occurred at a top drift (0.55%) which is lower than the fully infilled frame (1.3%) and the bare frame (2.6%).

Figure 6.28 : Vertical distribution of lateral displacement for case study-2 infilled Pilotis

Figure 6.29 : Capacity curve for case study-3 infilled Pilotis
6.7.4 DAP results for case study-4 infilled Pilotis

The case study-4 is a five storey frame following the Turkish practice of gravity load designed frames without seismic provisions (see section 3.4). The capacity curve is shown in Figure (6.31) which demonstrated a typical load-displacement relationship similar to the bare frame. The formation of a soft first storey is predicted according to the presence of a reduced stiffness at that storey as can be seen in Figure (6.32) which is the same behaviour as the case of the bare frame (see section 5.6.4) and the fully infilled frame (see section 6.6.1.4). The global yield step of the infilled pilotis frame occurred at a top drift (0.18%) which is lower than its value for the bare frame (0.3%). The collapse step of the infilled pilotis frame occurred at a top drift (0.7%) which is close to the case of fully infilled (1.0%) and bare frame (1.1%).
6.7.5 **DAP results for case study-5 infilled Pilotis**

The case study-5 is a seven storey frame following the Turkish practice of moment resisting frames with embedded wide beams frames with seismic provisions (see section 3.5). The pushover capacity curve is shown in Figure (6.33) which demonstrated a load-displacement relationship stiffer than the case of the bare frame and with higher peak strength but without a sudden strength reduction as the fully infilled frame. At the collapse step, flexural hinging occurred at the bottom of the first storey columns and at the top of the third storey columns as can be seen in Figure (6.34) which is considered a partial column sway mechanism. The behaviour is similar to the case of the fully infilled frame where partial column sway mechanism occurred between the first and the fourth floor levels (see section 6.6.2) and is different than the case of the bare frame where beam hinging pattern occurred (see section 5.6.5). The global yield step of the infilled pilotis frame occurred at a top drift (0.16%) which is lower than both the fully infilled frame (0.4%) and the bare frame (0.65%). The collapse step of the infilled pilotis frame occurred at a top drift (1.4%) which is lower than the case of fully infilled (1.6%) and the bare frame (2.1%).
6.7.6 DAP results for case study-6 infilled Pilotis

The case study-6 is a seven storey frame following the seismic design provisions of the Yugoslavian code 1984 of moment resisting frames (see section 3.6). The capacity curve is shown in Figure (6.35) which demonstrated a load-displacement relationship stiffer than the case of the bare frame and with higher peak strength. At the collapse step, soft first storey mechanism occurred (see Figure 6.36) where for the fully infilled frame a partial column sway mechanism occurred (see section 6.6.3.1) and a beam hinging pattern was observed in the case of bare frame (see section 5.6.6). The global yield step of the infilled frame occurred at a top drift (0.1%) lower than its value for the fully infilled frame (0.45%) and for the bare
frame (0.64%). The collapse step of the infilled pilotis frame occurred at a top drift (0.75%) which is lower than the fully infilled frame (1.6%) and the bare frame (3.2%).

Figure 6.35: Capacity curve for case study-6 infilled Pilotis

Figure 6.36: Vertical distribution of lateral displacement for case study-6 infilled Pilotis
6.7.7 DAP results for case study-7 infilled Pilotis

The case study-7 is a four storey frame following the seismic design provisions of the Portuguese code of moment resisting frames (see section 3.7). The capacity curve is shown in Figure (6.37) which demonstrated a load-displacement relationship stiffer than the case of the bare frame and with higher peak strength. At the collapse step, soft first storey mechanism occurred (see Figure 6.38) where for the fully infilled frame a partial column sway mechanism occurred (see section 6.6.3.2) and a beam hinging pattern was observed in the case of bare frame (see section 5.6.7). The global yield step of the infilled pilotis frame occurred at a top drift (0.16%) which is lower than its value for the fully infilled frame (0.3%) and the bare frame (0.83%). The collapse step of the infilled pilotis frame occurred at a top drift (1.16%) lower than its value of the fully infilled frame (2.0%) and the bare frame (3.3%).

![Capacity Curve](image)

**Figure 6.37 : Capacity curve for case study-7 infilled Pilotis**
Figure 6.38: Vertical distribution of lateral displacement for case study-7 infilled Pilotis.
7 DISCUSSION AND COMPARISON OF RESULTS

In this section, the sway potential indices, both the deformation based and the strength based are compared with predicted failure mechanisms resulting from the numerical simulation using the fiber element modelling software (SeismoStruct) by applying the displacement based adaptive pushover method. The target is to observe how far the prediction using the sway indices can capture the most probable failure mechanisms. The expected outcomes are the verification of both sway indices and defining the limits that control the expected behaviour as beam or column sway mechanisms in three cases of reinforced concrete frames, which are bare, fully infilled and infilled pilotis.

7.1 Discussion on the limits of the sway indices

7.1.1 Strength based sway index limits

According to the results of the pushover analysis prediction of the side-sway mechanism, the limits of the sway index given by Priestly (1997) for the strength-based sway index are updated, as shown in Figures (7.1),(7.2),(7.3). Each Figure show the relation between the normalized roof displacement for each frame and the corresponding sway indices values at each floor level. The normalized roof displacement is the ratio of the roof displacement to the storey displacement where the maximum inter-storey drift occurred at collapse step. For the case of bare frames, if the index is more than 0.85, a column sway mechanism is expected. For the case of fully infilled frames, if the index is more than 0.40, a column sway mechanism is expected. For the case of infilled pilotis frames, if the index is more than 0.1, a column sway mechanism is expected.
Chapter 7. Discussion and Comparison of Results

Figure 7.1: Strength based sway index limits for the case of bare frames

Figure 7.2: Strength based sway index limits for the case of fully infilled frames
7.1.2 Deformation based sway index limits

According to the results of the pushover analysis prediction of the side-sway mechanism, the limits of the sway index are defined, as shown in Figures (7.4),(7.5),(7.6). Each Figure shows the relation between the normalized roof displacement for each frame and the corresponding sway indices values at each floor level. For the case of bare frames, if the index is more than 1.5, a column sway mechanism is expected. For the case of fully infilled frames, if the index is more than 1.0, a column sway mechanism is expected. For the case of infilled pilotis frames, if the index is more than 0.5, a column sway mechanism is expected.
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Figure 7.4: Deformation based sway index limits for the case of bare frames

Figure 7.5: Deformation based sway index limits for the case of fully infilled frames
7.1.3 Correlation between the strength and deformation based sway indices

One of the main objectives of this research work is to develop a simple method of predicting the most probable failure mechanism of existing RC frames using the geometry which is related to the stiffness. By relating the stiffness to the strength, the deformation based index can be used alone to predict the failure mechanism which has the advantage of simple and direct evaluation and can be used for the assessment of large scale building stock. Figure (6.7) shows that the strength based and deformation based sway indices are well correlated. More emphasis on the fact that, the main idea is to substitute the "strength based" index with a "deformation based" one which will work as good or even as bad as the former.
Chapter 7. Discussion and Comparison of Results

7.2 Discussion on the Storey strength and stiffness ratios

A general observation from the results of the gravity load designed frames presented in the previous chapters is that they exhibited a column sway mechanism at the weakest storeys in the case of bare frames. Even with the values of the sway indices predicting column sway at all the floor levels, it is not enough to predict which storey level would exhibit the column sway mechanism. The controlling parameter in this case is the vertical distribution of the storey strength and stiffness along the height of the frame. It is suggested that another parameter should be added in the assessment procedures which relates the relative storey strength and stiffness as follows:

\[
K_{si} = \frac{V_{n+1}}{V_n}, \quad K_{di} = \frac{E_{n+1}}{E_n}
\]  

(7.1)

where \(K_{si}\) is the storey relative strength ratio, \(V_{n+1}\) is the storey strength above the floor level, \(V_n\) is the storey strength below the floor level, \(K_{di}\) is the storey relative stiffness ratio, \(E_{n+1}\) is the storey stiffness above the floor level and \(E_n\) is the storey stiffness below the floor level.

Another observation that could be useful to rely on the storey relative stiffness alone to predict the mechanism is shown in Figure (7.8) where the storey stiffness and strength are proportional especially at the lower region of the plot where the gravity load designed frames exist. In other words, when the strength is reduced with height, it is most probably that the
stiffness is reduced as well. The correlation between the storey strength and stiffness ratios is shown in Figure (7.9).

It should be noted that for the case of frames that are designed with seismic provisions which exhibit a beam sway mechanism, the storey strength and stiffness ratios are not required as additional information for the side-sway mechanism prediction.
Chapter 7. Discussion and Comparison of Results

7.3 Comparison and discussion of the gravity load designed frames results

7.3.1 Case study-1 discussion

The case study-1 represents a gravity load designed frame which exhibited a column sway mechanism for all cases with and without masonry infills at the first floor as explained by the sway indices where the beams are stronger and stiffer than the columns and it is observed that the lateral displacement capacity for all cases is nearly the same (see Figure 7.10). Additionally, the distribution of the storey strength and stiffness is uniform along the height as shown in Figure (7.11). There is no sudden change in the storey strength and stiffness ratios and the plastic deformation is concentrated at the first storey for the case of the bare frame. In the case of the fully infilled frame, it can be observed that if the sway indices predict column sway for the bare frame, the same behaviour is also expected for the fully infilled frames as the sway index at the first floor is much larger than the limiting value which in this case is about 5 times the limiting value for strength based index and about 2.5 times the limiting value for deformation based index.

![Figure 7.10 : Comparison of DAP results with sway indices for case study-1](image)

![Figure 7.11 : Relative storey strength and stiffness of case study-1](image)
7.3.2 Case study-2 discussion

For case study-2, which is a gravity load designed frame, the sway indices predicted a column sway mechanism for the bare frame at all storey levels but it is not possible to identify where the storey mechanism would occur (see Figure 7.12). The need for the storey strength and stiffness ratios is clear in this case, as it predicted well the storey mechanism at the second and the fourth storeys where there are sudden reductions in both strength and stiffness as shown in Figure (7.13). The storey strength ratios are 0.77 and 0.75, which are less than most of the code limits for vertical strength irregularities (see Chapter 2). The storey stiffness ratios are 0.51 and 0.65, which are also less than the code limits for vertical stiffness irregularities.

The effect of the masonry infills is clear in changing the failure behaviour of the bare frames to form a soft first storey mechanism regardless of the strength and stiffness distribution of storeys with height because the sway indices are much larger than the limiting index at the first floor level which are about 3 times this value for the strength based index and 2.5 times for the deformation based index. Thus, for the case of masonry infilled frames designed for gravity loads only, the sway potential index at the ground floor level is the key parameter in predicting the failure mechanisms. For the case of infilled pilotis frame, the expected behaviour of soft first storey is observed.

![Figure 7.12: Comparison of DAP results with sway indices for case study-2](image-url)
Chapter 7. Discussion and Comparison of Results

7.3.3 Case study-3 discussion

For the case study-3, which is part of the gravity load designed frames group, the sway indices predicted column hinging for the bare frame at the third floor level but they did not give any information about the type of side-sway mechanism, if it would be either partial or a soft third storey mechanism. The storey strength and stiffness ratios predicted well where the type of failure mechanism as the values of the $K_s$ and $K_d$ are equal to 0.60 and 0.63 respectively at the second floor level which show the sudden reduction of strength and stiffness at that level (see Figure 7.15), consequently, it was observed that the bottom sections of the third storey columns formed flexural hinges and a third storey mechanism occurred as shown in Figure (7.14). For the case of masonry infilled frame, the soft first storey mechanism occurred as predicted by strength based index which is 1.3 times the limiting value and the deformation based index which is 1.5 times the limiting value. The same behaviour was observed for the case of infilled pilotis frame.

Figure 7.14: Comparison of DAP results with sway indices for case study-2
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Figure 7.15: Relative storey strength and stiffness of case study-3

7.3.4 Case study-4 discussion

For the case study-4, which is part of the gravity load designed frames group, the sway indices predicted column sway mechanism at all floor levels and the DAP prediction was the formation of a soft first storey mechanism as shown Figure (7.16). The storey strength and stiffness ratios captured the expected result by the DAP analyses as the values $K_s$ and $K_d$ at the first floor level are larger than unity which reflects the sudden change in both strength and stiffness at that level (see Figure 7.17). It should be noted that the strength based index at the first floor is less than the limiting value 0.85 which is not expected when having taller first storey. Thus, the use of the strength and deformation sway indices is important along with the storey strength and stiffness ratios for better prediction of the failure mechanism.

For the case of masonry infilled frame, the same side-sway behaviour is predicted by forming a first soft storey mechanism. Thus, for the gravity load designed frames with taller first storey it is expected to have the same side-sway mechanism for the three cases of bare, fully infilled, and infilled pilotis.
Chapter 7. Discussion and Comparison of Results

Figure 7.16 : Comparison of DAP results with sway indices for case study-4

Figure 7.17 : Relative storey strength and stiffness of case study-4

7.4 Comparison and discussion of the embedded beam frame (case study-5) results

For the case study-5, which is an embedded beam frame, the sway indices predict the behaviour of the bare frame as a beam sway mechanism at all floor levels (see Figure 7.18); this is confirmed with the DAP analysis which showed a beam hinging pattern. There is no effect of the sudden reduction in strength and stiffness at the second and fourth floor levels in terms of the mechanism (see section 3.5), which shows that in this type of frame, there is no need to evaluate the storey strength and stiffness ratio for the case of bare frame.

For the case of fully infilled frame, a partial mechanism has been predicted by the DAP analysis between the second and the fourth floor levels which confirm with the prediction of the storey strength and stiffness ratios that show the sudden reduction of both values at the same floor levels as shown in Figure (7.19). For the masonry infilled pilotis frame, a partial
mechanism has been predicted by the DAP analysis between the first and the third floor levels. According to the previous observation, it can be concluded that for this type of frame, the sway indices limiting values are different from conventional frames and the failure mechanisms for the infilled frames are mainly dependent on the vertical distribution of the strength and stiffness along the height.

![Figure 7.18: Comparison of DAP results with sway indices for case study-5](image1)

![Figure 7.19: Relative storey strength and stiffness of case study-5](image2)
7.5 Comparison and discussion of the seismically designed frames results

7.5.1 Case study-6 discussion

For the case study-6, which represents a RC frame with seismic provisions, the sway indices predict beam sway mechanism; this is confirmed with the DAP analysis for the case of the bare frame (see Figure 7.20). Even with the presence of a sudden reduction in strength and stiffness at the first floor level which is reflected by the value of $K_s$ and $K_d$, it did not affect the overall side-sway behaviour (see Figure 7.21). This can be explained by the values of the sway indices which are much lower than the limiting values. The strength based index is 0.5 times the limiting value and the deformation based index is 0.75 times the limiting value. For the case of masonry infilled frame, the predicted mechanism by DAP analysis is partial mechanism up to the third floor level where the sway indices are below the limiting values for both strength and deformation based. For the infilled pilotis frame, the expected behaviour is the soft first storey mechanism.

![Figure 7.20: Comparison of DAP results with sway indices for case study-6](image1)

![Figure 7.21: Relative storey strength and stiffness of case study-6](image2)
7.5.2 Case study-7 discussion

For the case study-7, which represents a RC frame with seismic provisions, the sway indices predict a beam sway mechanism that was also observed from the DAP analysis for the case of the bare frame as shown in Figure (7.22). The storey strength and stiffness ratios for all floor levels are within the limits provided by codes as shown in Figure (7.23).

For the masonry infilled frame, the partial mechanism up to the second floor level is observed using DAP analysis. The limiting value of 0.4 for the strength based index and 1.0 for the deformation based index are exceeded at all floor levels which predict a column sway mechanism.

![Figure 7.22: Comparison of DAP results with sway indices for case study-7](image)

![Figure 7.23: Relative storey strength and stiffness of case study-7](image)
8 CONCLUSIONS

The motivation for the presented research work is the need for a rational and reliable simplified assessment method for predicting the most probable side-sway failure mechanism for existing RC frame buildings, which is a basic step in evaluating the seismic capacity of these buildings.

In order to predict the most probable failure mechanism, the concept of the sway potential index is introduced. A strength based sway index suggested by Priestley (1997) as well as a newly proposed deformation based index are verified. The concept of the strength based index is whether plastic hinges occur in beams or columns by comparing the sum of the probable flexural strengths of the beams and the columns at the joint centroids at a horizontal level. The present study verified the suggested index and updated the limiting values between beam sway and column sway for different types of existing RC frame buildings as shown in Figure (8.1).

![Figure 8.1: Strength based sway index limits](image-url)
Chapter 8. Conclusions

The deformation based sway index is introduced considering the relative flexibility of beams and columns using only the geometrical properties as an indicator of potential collapse mechanisms. The definition of the most probable side-sway mechanism of the frame is evaluated by comparing the yield rotation of the columns to those of the beams. The new index is a measure of the dominant flexible members that are expected to control the behaviour of the frame.

The present study verified the suggested index and updated the limiting values between beam sway and column sway for different types of existing RC frame buildings as shown in Figure (8.2).

![Figure 8.2: Deformation based sway index limits](image)

According to the results obtained using numerical modelling, specifically for the gravity load designed bare frames, the prediction using the sway potential indices alone can only give the overall side-sway mechanism as a column sway without defining which storey would exhibit the predicted column hinging pattern. The study introduced a storey strength and stiffness ratios to overcome this difficulty, which reflect the vertical distribution of the strength and stiffness along the height of the frame. The concept of the storey strength and stiffness ratios is to compare the shear strength and stiffness at a storey level to the one above or below. The limits for these storey ratios are verified as presented in Figure (8.3). In the case of the bare frames, the allowable range for the storey strength ratio ($K_s$) is $\pm 20\%$, and for the storey stiffness ratio ($K_d$) is $\pm 30\%$. 

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Figure 8.3: Storey strength and stiffness ratios allowable range for bare frames

By applying the previous limits to the case study frames, conclusions regarding the applicability of the concept of the sway potential index can be summarized in the following points.

- For the case of gravity load designed RC bare frames, the most reliable method to predict the side-sway mechanism is to use either the strength based sway index (Sis) with the storey strength ratio (Ks) or the deformation based sway index (Sid) with the storey stiffness (Kd) ratios. These combinations are reliable as a result of that the strength and the stiffness in that type of RC frames are proportional.

- For the case of gravity load designed RC bare frames, with sway potential indices predicting column sway for all floor levels, and the values of the (Ks) and (Kd) are within the allowable range, then it is expected to have a soft first storey mechanism. If the (Ks) and (Kd) values are not within the allowable range at any floor level, it is expected to form a storey mechanism at these floor levels or a partial column sway mechanism with more than one storey where a sudden change of strength and stiffness occur.

- For the case of gravity load designed RC fully infilled frames, with sway potential indices predicting column sway for all floor levels, it is expected to have a soft first storey mechanism regardless of the values of (Ks) and (Kd).

- For the case of RC bare frames with seismic design provisions, the sway potential indices predict beam sway for all floor levels, and the values of the (Ks) and (Kd) are within the allowable range. Accordingly, the sway potential indices are sufficient indicators of the most probable side-sway mechanism.

- For the case of RC infilled frames with seismic design provisions, the sway potential indices are sufficient for predicting the sides sway mechanism and it is expected to form a partial mechanism between the ground floor and higher storey levels where the sway index values becomes more than the limits for beam sway.
Chapter 8. Conclusions

- For the case of RC bare frame with embedded beams, the flexible beams hinging dominates the side-sway mechanism. The case of fully infilled frame has different response by forming a partial mechanism. This type of frames should have different limiting values for the sway indices and need further studies to predict the most probable mechanism.

- For the case of RC infilled Pilotis frames with or without seismic design provisions, it is expected to have a soft first storey mechanism. The only case that is different is the frame with embedded beams where a partial mechanism may form.

- The newly proposed deformation based sway index along with the storey stiffness ratio are expected to be used reliably to predict the side-sway mechanism where they have the advantage of being calculated using only the geometry of the frame according to the expected proportionality between the strength and stiffness. This combination is mainly applied to the existing gravity load designed frames with no seismic provisions that represent a large ratio in the building stock in most of the seismically vulnerable countries and which have a large variability in their geometrical and material properties.

- Future research work is expected to take into account the effect of the ground motion intensity, and the associated variability in the activation of the most probable side-sway mechanism, especially for the case of masonry infilled RC frames.
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APPENDIX A:

Table A 1. Flexural strength of beam sections for case study-2

<table>
<thead>
<tr>
<th>Beams</th>
<th>Reinforcement (mm$^2$)</th>
<th>Reinforcement Ratio %</th>
<th>Flexural Strength My (KN.m)</th>
<th>Tension side</th>
</tr>
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<tbody>
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<td></td>
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<td>Bottom</td>
<td>Top</td>
<td>Bottom</td>
</tr>
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<td></td>
<td>804.25</td>
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<td></td>
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<td>402.12</td>
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Table A.1: Flexural strength of column sections for case study-2

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<th>Axial force (KN)</th>
<th>Axial Load Ratio %</th>
<th>Reinforcement (mm$^2$)</th>
<th>Reinforcement Ratio %</th>
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Table A.2: Flexural strength of beam sections for case study-3

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<th>Tension side</th>
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Table A.3: Flexural strength of column sections for case study-3

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### Table A.4: Flexural strength of beam sections for case study-4

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<th>Tension side</th>
</tr>
</thead>
<tbody>
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<td>Top</td>
<td>Bottom</td>
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<td>0.68</td>
</tr>
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### Table A.5: Flexural strength of column sections for case study-4

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<th>Axial Load Ratio %</th>
<th>Reinforcement (mm$^2$)</th>
<th>Reinforcement Ratio %</th>
<th>Flexural Strength My (KN.m)</th>
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Table A.6: Flexural strength of beam sections for case study-5

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<th>Tension side</th>
</tr>
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<td></td>
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Table A.7: Flexural strength of column sections for case study-5

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<th>Reinforcement (mm2)</th>
<th>Reinforcement Ratio %</th>
<th>Flexural Strength My(KN.m)</th>
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### Table A.8: Flexural strength of beam sections for case study-6

<table>
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<th>Beams</th>
<th>Reinforcement (mm²)</th>
<th>Reinforcement Ratio %</th>
<th>Flexural Strength My (KN.m)</th>
<th>Tension side</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top</td>
<td>Bottom</td>
<td></td>
<td></td>
</tr>
<tr>
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<td>567.06</td>
<td>0.39</td>
</tr>
<tr>
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<td>850.59</td>
<td>567.06</td>
<td>0.59</td>
</tr>
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<td>567.06</td>
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### Table A.9: Flexural strength of column sections for case study-6

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<th>Axial Load Ratio %</th>
<th>Reinforcement (mm²)</th>
<th>Reinforcement Ratio %</th>
<th>Flexural Strength My(KN.m)</th>
</tr>
</thead>
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### Table A.10: Flexural strength of beam sections for case study-7

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<th>Tension side</th>
</tr>
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</tr>
<tr>
<td>1st Floors</td>
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<td></td>
</tr>
<tr>
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<td>0.29</td>
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<tr>
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<td>end_2</td>
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<td>0.32</td>
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### Table A.11: Flexural strength of column sections for case study-7

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<th>Reinforcement (mm²)</th>
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