CHAPTER 1: INTRODUCTION

This research work has been done to improve the structural behaviour (*affected by the buckling of the members*) of the conventional steel braced frames. The braced frames that were constructed before the development of seismic provisions have been referred in the research articles as the non-ductile conventional steel braced frames, non-ductile concentric steel braced frames or the non-seismic steel braced frames (NCBFs). As per most of the research articles, the braced frames constructed before 1988 have been considered in this category. These braced frames have been found to be deficient and lack the important structural features (*regarding the design and construction procedures*) that have been considered necessary in the current seismic provisions and the steel design specifications (*to resist the severe seismic loads or even the moderate seismic loads*).

The NCBFs have not been found to be severely deficient that they couldn't be upgraded; as, not all the members of these braced frames were deficient. Due to the use of low toughness electrode welds, the welded connections were little deficient and few structural components like the slender braces and the weak beams (*in case of Chevron braces*) were found to be geometrically deficient. In most of the cases, the cross-section of columns has been found to be significantly sufficient to comply with the present codal provisions.

The other face of the structural behaviour/ capabilities of the older NCBFs has been their design considerations, which were significantly conservative (*very high factor of safety*). This positive side of these NCBFs could allow them to be a good candidate for the renovation. The recently constructed ordinary concentrically braced frames, OCBFs have also been found to resist only the moderate seismic loads. So, the OCBFs could also be considered as a candidate to be upgraded using the proposed renovation strategies.

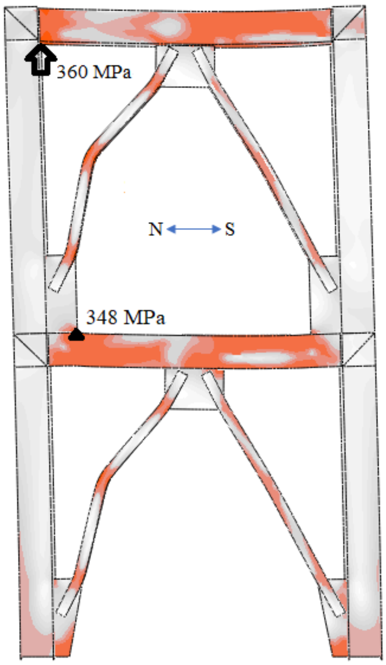


Figure 1.1 Deformed state of the chevron (*Ch*) braced frame

In general, the braces have been used to provide resistance against the horizontal/lateral loads induced due to the external forces. In the experimental study by Wakabayashi et. al. (1967), it was found that the X-braced frames behaved quite similar under monotonic or cyclic loading, i.e., the frame members (*beams and columns*) were not very much affected by the event of buckling of the braces.

In the same study, structural behaviour of the chevron braced frames under incremental cyclic loading was found to be such that after the buckling of the braces, unbalanced forces (*due to loss of strength of brace in compression*) started acting on the beam. These unbalanced forces at the beam-brace connection, resulted into serious deflection of the beam, which in turn caused the remaining brace (*supposed to resist the lateral load through tensile action*) to become ineffective. So, the chevron braces were found to experience serious strength degradation due to buckling of the braces. The deformed state of the experimented specimen numerically analyzed on the Abaqus CAE software (2017), has been shown in Figure 1.1 (*darker the colour shade, higher the Von-Mises stress*).

When a structure/member of a structure is subjected to compressive load, buckling of them may occur. Buckling is characterized by a sudden sideways bowing of the member, giving rise to large deformation. Buckling is a case of geometrical instability which doesn't depend on the yield strength of the structure. As shown in Figure 1.2, buckling might occur even if the stresses developed in the structure (σ_c) were well below yield strength of the material (σ_y). As long as the load is small, axial shortening of the member can be observed. Once the certain critical load is reached on increasing the applied compressive load (P) on a member, it may ultimately become large enough to cause the member to become unstable and it is said to have buckled. Further loading will cause significant and somewhat unpredictable deformations; theoretically, leading to the complete loss of the member's load-carrying capacity under compression.

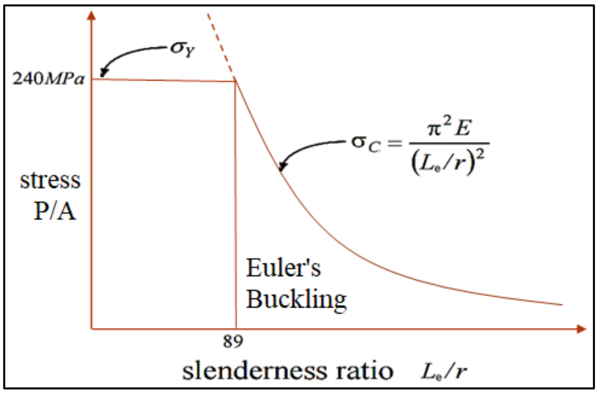


Figure 1.2 Critical stress vs slenderness ratio (Steel)

In 1757, mathematician Leonhard Euler derived a formula that gives the maximum axial load that a long, slender, ideal column (*perfectly straight, made of a homogeneous material and free from initial stress*) can carry without buckling. When the applied load reaches the Euler load, called the critical load, the column comes to be in a state of unstable equilibrium. The introduction of the slightest lateral force at that load will cause the column to fail by suddenly "jumping" to a new configuration, and then column is said to have buckled. Its mathematical derivation has been shown in the Figure 1.3.

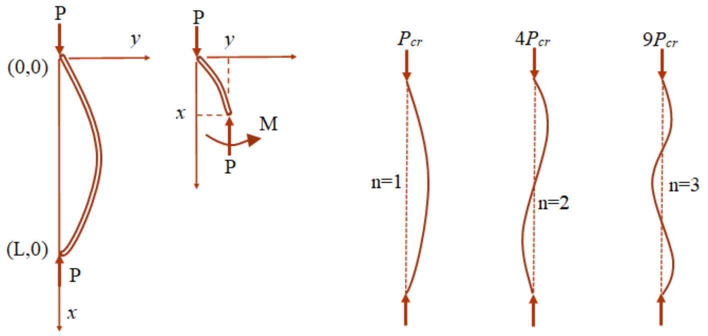


Figure 1.3 Derivation of classical Euler buckling load, modes

The equations derived based on Figure 1.3 have been given below; where, '*P*' is the applied critical load at which buckling (P_{cr}) may occur, '*E*' is elasticity modulus, '*I*' is moment of inertia, '*L*' is total length and '*n*' is modes of buckling.

$$EI\frac{d^2y}{dx^2} + Py = 0, y(0) = 0, y(L) = 0$$

$$y^{''} + \lambda y = 0, y(0) = 0, y(L) = 0, \lambda = P/EI$$

$$P_n = \frac{n^2\pi^2 EI}{L^2}, n = 1,2,3 \text{ etc.} \qquad \text{Equation 1.1}$$

After experimental observations, different values of effective length coefficient (*K* = *effective length/ total length*) for the rigidly connected braces obtained in various past researches were as follows, *K*= 0.5 for chevron brace (Wakabayashi and Tsuji 1967); *K*= 0.8 for chevron brace (Trembley 2000); *K*=0.5 for single diagonal brace and *K*=0.6 or 0.7 for in-plane or out-plane buckling of X-brace respectively (Wakabayashi *et al*. 1980); *K*= 0.5 or 0.7 (*close to theoretical*) for in-plane or out-plane buckling of braces respectively (Roeder, 1989); *K*=0.51, if the connected member had section size much higher than the buckling member and 0.63 (*close to design value of K=0.65, given in IS:800 (2007) and JSCE (2009)*), if the connected member had section similar to that of the buckling member (Narayan *et al*. 2020). The effective length values considered for the design in compliance with IS: 800 (2007) has been shown in Figure 1.4.

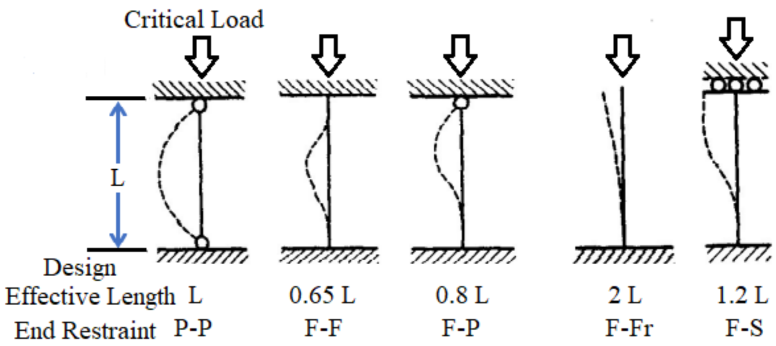


Figure 1.4 Buckling of strut for various boundary conditions (IS:800 2007)
Note: *P, F, Fr* and *S* respectively represents pinned, fixed, free and slider end constraints.

This critical load is the limiting load below which axial compression in an unbent configuration is possible. The transition from straight to bent configuration takes place because at the buckling load straight configuration ceases to be stable. So, the buckling load marks the limit of stability. The essence of buckling is the disproportionate increase in displacement as a result of a small increase in load. This implies that buckling analysis is a subtopic of nonlinear mechanics. Stress and strains are assumed to obey hooks law; therefore, this nonlinearity is purely geometrical. So, ascertaining the working of the structure by preventing buckling of its members is very complex.

Wakabayashi *et. al*. (1977) found that bringing the slenderness of the braces less than 30 had detrimental effects on the behaviour of the braced frames under repeated lateral loads and Narayan and Pathak (2020) found that the bringing the slenderness close that of beams and columns had a detrimental effect on the columns (*analysed using linear buckling analysis*). Here, utilizing Equation1.1, rather than increasing the cross-section (I) of the braces, the concept of multi-node (n) buckling was adopted to increase 'P_{cr}'.

1.1 PROBLEM STATEMENT

The centrically braced frames (*CBFs*) having chevron type braces were quite popular in olden days because of their aesthetic appearance and the availability of the ample space for the openings. The problem with these braced frames (*including the ordinary* *concentrically braced frames, OCBFs having chevron braces that were constructed later*) was that under the effect of repeated/ seismic loading, the braces would lose their strength in 'compression' after buckling at once. Such brace would provide negligible resistance against compression in the next loading cycle. This situation would give rise to an unbalanced force on beam at the concentric junction of the braces. Because of this unbalanced force, the beam would undergo local buckling and significant vertical deflection which would lead to the severe strength degradation of the overall braced frame. This vertical deflection of beam would also result in a very undesirable behavior, that would be the ineffectiveness of the remaining brace resisting the lateral loads by undergoing tension. The eccentric braces have been considered to be one of the simplest and the most effective ways to dissipate the inelastic energy under seismic loading but in olden chevron braces the eccentricity was not provided from the structural engineering perspective rather it was provided just for the aesthetic purpose and to provide more opening space. So, the non-ductile steel brace could be in the form of an eccentrically braced frame, which wouldn't be able to provide the desired beneficial structural behaviour (*observed significant beam deflections and undesirable brace deformation*).

Even the desired working of eccentric braces to dissipate energy is quite destructive (*only the link attached to beam has to dissipate energy by undergoing excessive shear/ rotation down the slab*). It would not be less destructive if some part of energy was dissipated by the other members without landing into the condition of strength degradation.

In single diagonal and cross braced frames, the condition of unbalanced forces was not found. But they been found to provide limited lateral load resistance under the moderate seismic forces due to the loss of compressive strength after buckling of the brace and the only remaining resisting component being the tensile one.

1.2 OBJECTIVE AND SCOPE

In particular, the braced frames have been usually referred as primary seismic force resisting systems (*SFRS*). It would be understood from this study that all the braced frames substantially increase the vertical load capacity. But, the main purpose of the bracing has been to provide lateral load resistance, which could be affected severely by the occurrence of the buckling of the braces and would result into an ineffective configuration of bracing. The theoretical aspects of buckling have already been studied by various researches (*Lokkas 1996 elaborately explained about the buckling design analysis of rigid-jointed steel frames*) and have been explained in various books (Chajes 1974; Brush and Almroth 1975) to a great depth. Concepts of finite element analysis (*FEM*) have been well explained in the book by Chandrupatla & Belegundu (1991).

So, in this research, rather than focusing on the theoretical aspects of buckling; objective of this research work was to devise the renovation strategies/ techniques to overcome the problems encountered in the older steel braced frames due to the buckling (*from both the strength/stability and ductility perspectives*) of the members. Under the scope of this work, older steel braced frames like the prominent conventional braced frames and a truss bridge were considered for the renovation/upgrade to avoid the problems related to compression/ buckling. To ascertain strength and stability against buckling, linear perturbation buckling analyses were conducted and to ascertain the ductility, non-linear quasi-static analyses were carried out.

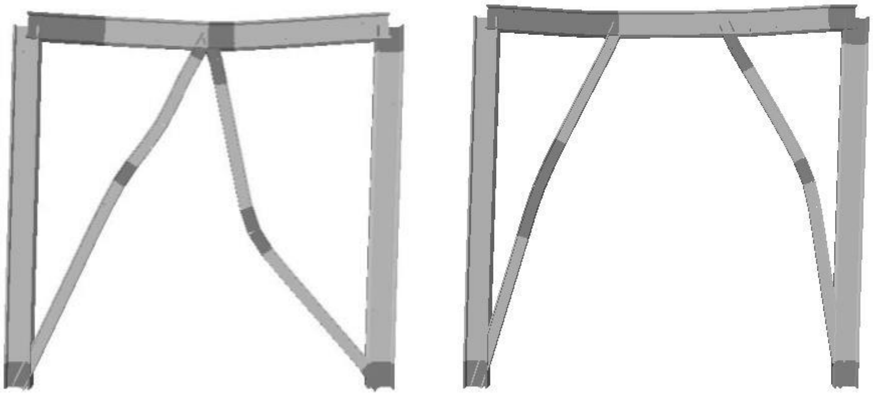
Major considerations in devising the strengthening measures have been listed below:

1. The strategies should be design friendly such that they could be implemented in compliance with most of the available seismic codes or steel design specifications.

2. The renovation strategies should be very economical. Complete replacement of the functional structures in wake of increasing loads could be avoided.
3. Renovation strategies could be implemented for fresh design of braced structures.
4. In the modification procedure, no replacement of main structural components should be required and the aesthetics of the structures should be least hampered (*important for the ones that are of historic importance*).
5. This would cause least harm to the environment, would require minimal structural intervention and would not be disruptive to the occupants/users of that structure.
6. The strategies would be handy (*easy to execute*), will have minimal requirement of the skilled labour and the procedures would be quite simple to communicate.
7. The additional material take-off would be kept minimal and the requirement of heavy machinery and equipment would be minimal or conditional.
8. Most importantly, the strategies would be capable of overcoming the problems encountered due to the buckling of members of the steel braced frames and would make them structurally sound (i*mprove the parameters like load bearing capacity, elastic-state reattainment of the main frame, hysteretic behaviour, higher energy dissipation and higher deformation capacity and reduced stress concertation etc.*); after that they would be capable to resist the forces higher than their design loads; would resist seismic forces higher than the moderate seismic forces.
9. After the proper implementation of the proposed strategies, the accommodation or the work spaces would surely provide a safer habitat to the occupants/ users.

1.3 INSIGHT ABOUT THE STEEL BRACED FRAMES

1.3.1 Concentrically braced frames (CBFs): In this type of braced frames, braces are connected in such a manner that their connections are concentric. Here, the braces are supposed to be the main energy dissipating members. Strength degradation after buckling of braces is one of the detrimental phenomena observed in them.



a) b)

Figure 1.5 Deformation of concentric (*Ch*) and eccentric (*Che*) chevron NCBFs

For the 'chevron' CBFs, where the beam deflection can be severe after buckling of its connected brace, such behaviour has been called as unreliable and uncertain (Popov *et. al.* 1987; Popov 1983). The deformed shape of one such NCBF has been shown in Figure 1.5 (a). Uncertainty in chevron CBFs in terms of residual '*post-buckling*' compressive strength (*RPBCS of the braced frame, not of an isolated brace; the compressive strength of an isolated brace would be completely lost after buckling*) has been discussed below.

- Based on various experimental literatures, according to Sabelli (2001) a typical brace demonstrates a *RPBCS* of 30% to 50% of its initial buckling strength.
- Seismic provision of AISC (2016) used 30% of the nominal buckling strength for RPBCS for the design of such braced frames.
- Kim *et. al.* (2017) used 20% *RPBCS* of the nominal buckling load.
- Trembley et al. (2000) used 20% of the nominal buckling load as the *RPBCS*.

- Rai *et. al.* (2003) used 55% of the nominal buckling load as the *RPBCS*.
- European code, EN-1998, completely neglected the contribution of compression chord (*very conservative*).
- Kanyilmaz (2017) found that the contribution of compression chord to the 'global' stiffness was 38 to 54%. During post-buckling stage, contribution to the global elastic resistance was found to be 16 to 32%. After initiation of net section fracture, contribution to overall resistance was found to be 18 to 23%.

1.3.2 Eccentrically braced frames (EBF): In this type of braced frames, off-set is provided between the ends of braces either at the beam end or at the connection end. The energy dissipation has to be done by the link rotation/ shear and all other members including the braces should be strong enough to have minimal inelastic activity. In olden days, eccentricity was given just for the aesthetic/opening purpose; so, the ideal nature of EBFs couldn't be expected from such EBF-NCBFs, as shown in Figure 1.5 (b).

1.3.3 Division of braced frames based on seismic zone and construction era

*NCBF*s: They are the older braced frames, conventional braced frames. Non-ductile concentrically braced frames, (NCBFs) was the term used by Rai *et. al.* (2003) and Sen *et. al.* (2016). Non-Capacity Designed Braced Frames, Non-Seismic Concentrically Braced Frames were the term used by Sloat (2014). Existing older braced frames and pre-1988 CBFs were the term used by Sen *et. al.* (2019).

Low-ductility braced frames (*for moderate seismic zones*):

Low-ductility braced frames for use in moderate seismic zones include Ordinary Concentrically braced frames (*OCBFs*, R=3.25, *where R is the response reduction factor*) and R=3 braced frames introduced by AISC in 1997.

Understanding the 'R' factor:

Response reduction factor (R) given in IS: 1893 (2016) is as given below:

- R=3 for OMRF (Ordinary moment resisting frames)
- R=4 for OCBF (*Whereas in AISC, R value of braced frames can be as low as 3 for moderate seismic zones*)
- R=4.5 for Special braced frames, SBFs with concentric braces (*SCBF*)
- R=5 for SBF with EBF (*EBFs can only be used with SBFs*)

Design base shear (Design lateral force), $V_b = A_h \times W$ Equation 1.2

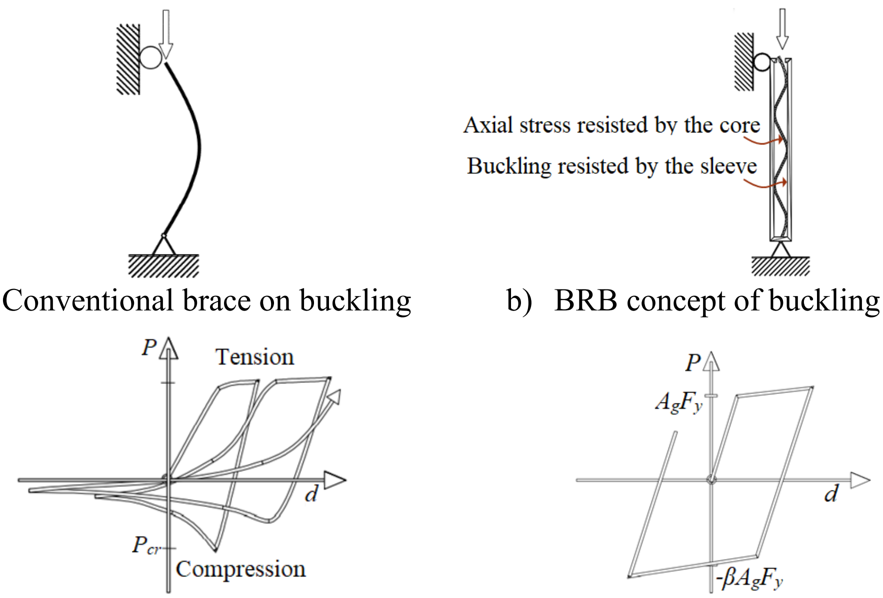
Where in Equation 1.2, W = Seismic weight of the building/ frame, A_h = Design horizontal seismic coefficient for structure; $A_h = (Z/2) \times (S_a/g)/((R/I)$. Again, in formula of A_h come the term R (*Response reduction factor*), Z = Seismic zone factor, S_a/g = Design acceleration coefficient for various soil types, I= Importance factor. The work presented here was performed by keeping in mind, the existing (*already constructed*) conventional braced frames (NCBFs) to be analyzed for buckling. So, the above-mentioned concepts of the seismic design for the construction of new ductile braced frames were not utilized here. After conducting linear perturbation buckling analysis on the NCBFs, non-linear quasi-static analyses were performed on them.

ASCE (2013) exempted R=3 steel seismic force resisting systems (*SFRSs*) in moderate seismic regions (Seismic Design Categories B and C) from seismic detailing and proportioning requirements like local (*width to thickness ratio*, *b/t*) and global (*KL/r*) slenderness limits for the braces and the consideration of amplified seismic loads for selection of beams, columns, and brace connections. R=3 OCBFs offer almost the same force reduction as the steel ordinary concentrically braced frame (*OCBF*) system, which has R = 3.25 (*not exempted from seismic detailing and proportioning requirements*).

Braces for high seismic hazard regions (*SCBFs, EBFs* and *BRBs*)

SCBFs (*Special Concentrically braced frame*) and EBFs: Seismic detailing and proportioning requirements are to be followed. Beams are made stronger to carry the unbalanced forces induced at the junction of the ends of the concentric braces and mid-section of the beams. In EBFs, the link (*length between the off-set of brace connection*) at the beam section which acts as a fuse (*dissipate energy by inelastic deformation*).

Based on the report of Sabelli *et. al.* (2004), the design concepts for the BRBs were inducted in the AISC seismic provisions. Sabelli *et. al*. followed some of the basic research concepts of BRBs given by of Watanabe *et. al.* (1988), Yoshida, K *et. al.* (1999, 2000). These concepts focused on the use of concrete infill in the steel sections. Whereas, nowadays the other types of BRBs have been developed like all-steel BRBs (*one type of all-steel BRB was also developed in this research work*). The characteristics of normal brace and a BRB have been explained through pictorial depiction in Figure 1.6.



a) Conventional brace on buckling

b) BRB concept of buckling restraint

c) Hysteresis curve for conventional brace/ strut

d) Hysteresis curve for BRB (*balanced and stable*)

Figure 1.6 Concept of conventional brace (as strut) and BRB (referred Sabelli 2006)

1.4 LITERATURE REVIEW AND LITERATURE GAP

Wakabayashi *et. al.* (1967, 1973, 1977-I, 1980-II, II) did pioneering works in assessing/ investigating the structural behaviour of the steel braced frames in a very detailed manner. Research works done by Minoru Wakabayashi have been the most inspiring force for the applicant to continue the research in this field of structural engineering. Development of the eccentrically braced frames have been quite a different section of the steel braced frames, which was significantly developed by Egor Popov. The research works of Popov *et. al.* (1976, 1983, 1987, 1988) have been the basis for the popularity of the eccentrically braced frames as one of the best primary seismic force resting systems. These research works formed the foundation for the development of the design provisions regarding eccentrically braced frames in almost all the seismic provisions throughout the world.

To experimentally analyze the failure patterns in the braced frames, forensic analysis of the link fracture in the eccentrically braced frames and the experimental investigation of buckling and fracture of braces in CBFs were done by Kanvinde *et.al.* (2009, 2015). Roeder (1989) has been doing pioneering research in the same area but in developing the important design criteria for the selection of the connections in various types of steel braced frames. He also gave the balanced design procedure for the design of gusseted connection, which is quite common in the design of connections. The work of Roeder has been followed by his students, fellows and co-authors including Lehman, Berman, Ballard, Sen, Sloat, Sizemore and Fahnestock etc. Sizemore *et. al.* (2017), Sen *et. al.* (2014, 2016) did experimental studies on NCBFs, specifically related to the deficiencies of the connections and the effect of the presence of weak beams on the overall structural behaviour of the steel braced frames. Sloat (2014) did an inventory on the presence of older steel braced frames (NCBFs) around/in Washington, USA.

Important points to refer from the inventory by Sloat, (2014) on NCBFs:

- Chevron braced frames were prominent (Architecture/space consideration)
- Hollow steel section (HSS), Wide flange sections (WF) sections were mainly used
- Not designed using seismic considerations, non-ductile

In present times, one of the best primary seismic forces resisting systems are the BRBs (*buckling restrained braces*). The work of Sabelli (2001, 2006) propagated the use of BRBs such that these works were the basis for the inclusion of the design specifications of BRBs in the AISC codes. Hybrid buckling-restrained braced frames and other methods of improving the structural behavior of the framed structures by using dampers have been developed by Charney *et. al.* (2008, 2014).

1.4.1 Solutions available for the retrofitting/ upgrade of the existing NCBFs:

a) Sen *et. al.* (2016): Replacing the connections, strengthening welds, concrete-filling in the hollow tubular section braces, replacing the existing braces with SCBF compliant braces or by replacing with BRBs (*popular these days, very skilled labour, costly, pre-fabricated in labs/factories*), which included the dismantling of the connections, serious structural intervention like removal of the slab (*also the beams*). They require high investment, serious structural interventions and disruptions to the occupants. Such retrofitting techniques were referred as severely disruptive to the occupants and have been found to demand serious structural intervention as per Rai and Goel (2003).
b) Rai and Goel (2003): Several strategies were suggested to upgrade the actually existing steel braced frame building (*having concentrically braced frames*) which included the concrete-filling of the hollow tubular section braces, replacement of existing beams with the beams that satisfy the seismic provisions specified for the

special concentrically braced frames (SCBFs) and complete replacement of the bracing configuration with the two-story X-braced bracing configuration, which required serious structural intervention.

To avoid the problems encountered in methods of upgrade or retrofitting of steel braced brace by Rai and Goel (2003) and Sen *et. al.* (2016), two methodologies for the upgrade/ renovation of the concentric and eccentric chevron braced frames have been suggested by Narayan and Pathak (2021, 2022). This was done by including additional diagonal/ vertical members attached to the existing braces and by generating multi-level eccentric chevron braced frames, MLEC-braced frame (*explained in the non-linear analysis section of this Thesis*) out of the existing braced frames *(both concentrically and eccentrically braced*). These strategies would cause minimal structural intervention and least disruption to the occupants, without replacement of any existing component of the braced frame.

It was found that on connecting of vertical and diagonal members from the existing brace to the frame and on converting into MLEC-braced frames, the structural behaviour of both the concentrically and the eccentrically braced frames was significantly improved. After the implementation of the previous two strategies, the buckling of the braces in the concentrically braced frames was still the cause of strength degradation and the sudden peaks observed in the hysteresis curves after initial buckling of the braces. In the later study, to improve the hysteretic behaviour of the concentric chevron braced frames, the chevron braces were converted into a new type of bucking restrained brace (BRB), stiffened-casing dual-sleeve BRB (SC-DS BRB). This BRB configuration was able to overcome the problem of sudden peaks observed in the hysteresis curves and to avoid the strength degradation along with reducing the concentration of excessive stresses.

1.4.2 Understanding the behaviour of steel braced frames

Popov (1983) illustrated using a graphical figure that major component of storey drift in MRFs is majorly contributed by rotation of floor beams, and to achieve the desired drift limitation the size beam has to be increased. Therefore, for functional reasons, use of diagonal bracing becomes more economical and safe option. If for moderate loads such braces behave in the elastic range, they provide an economical solution.

Popov *et. al.* (1987) focused on importance of links which act as fuses to prevent buckling of braces and provide ductility to the frame. EBF, CBF and MRF were compared for influence of link length (*e*) on the stiffness of frames. At '*e*' equal to the length of the beam, MRF would be obtained and it has minimum stiffness. As the length of link decreases, rapid increase in elastic stiffness occurs and the maximum stiffness would be obtained at $e = 0$, i.e., for the CBF condition. But the behaviour of CBF was considered unreliable under severe cyclic loading, so this condition was advised to be avoided.

Robert (2000) explained that the unbalanced forces in the beam after a brace had buckled can lead to soft storeys in the multi-storeyed frames. To overcome such condition, 2,4,8 and 12 storied chevron steel braced frames were designed with strong beam (*to carry the extra forces that are expected due to unbalanced brace forces*).

Sabelli (2001) explained about the procedure to estimate the brace deformation demand. Whole process of deformation under lateral loading was explained using pushover plot between fraction of buckling base shear vs. ductility for single storeyed frame.

Marstellar *et. al.* (2002) provided a handy chart for low rise buildings having chevron bracing, to estimate the bracing size and connection material required to resist lateral forces due to gravity, wind and low seismic loads (*seismic response modification factor equal to or less than 3*) designed for a given force in the braces (*with various conditions*).

Gengshu *et. al.* (2002) studied the method of determining the effective length of column and introduced a concept of threshold stiffness. Criteria for identifying a weakly braced frame were explained from various perspectives.

Hernandez *et. al.* (October 2008) considered 26 buildings having 4,8,12 and 16 storeys for 5 bays with chevron bracings at 2nd and 4th bays as external configuration and only at central bay as internal configuration. Minimum ratio of columns of moment frames was from the perspective of strong-column weak-beam weaker-brace collapse mechanism of bracing system. A relationship between collapse mechanism, building height and beam stiffness was also established. Pushover analyses of modes were carried using Drain-2DX computer program. One side of braced bay was found to be in tension with axial extension while other side was found in compression with axial shortening. Models where columns resist near 25%, 50%, 65%, and 75% of the total seismic shear forces were identified.

Eghtesadi (2011) selected 3, 6, 9 and 12 storied frames each being 4 bays wide with 2 corner bays braced. X, concentric/ eccentric chevron and diagonal bracing types were considered for examining the weight of the structure, top storey displacement and energy absorption. X- brace showed maximum energy absorption but this system was considered to have only one lateral force resisting member in each span. It implied that in comparison to other bracing systems it was found to have lower rigidity, so it required stronger beams and columns which would result in heavy weight of structure. Whereas in this study, high energy absorption, less requirement of steel-offtake and lower total weight was found to be achieved by chevron CBF system.

Landolfo *et. al.* (2011) quoted that restraining the braces from buckling allows symmetric response under tensile or compressive forces and enhances ductility significantly. Here more focus was given on reforming European design guidelines. According to Euro code, limit state of compression buckling of brace must be precluded at forces and deformations corresponding to 1.5 times the design storey drift. Using flexible MRFs in combination with BRBFs was said to provide re-centring capability along with significant post-yielding stiffness.

Amini *et. al.* (2012) included concentric X, V and chevron braces in 3, 5 and 7 storied buildings having 4 and 6 bays. Modelling was done using RAM-perform 3D software (Powell 2000). Two patterns of bracings were used, one being the bracing of two central bays (*adjacent*) and other, bracing of two end corner bays (*non-adjacent*). For all bracing types non-adjacent bays were found to have lower stiffness but higher strength in comparison to adjacent central bays configuration. Chevron brace was found to have higher stiffness. Ultimate resistance of chevron was found 50% higher than X-brace and hence concluded that using same value for response modification factor was not appreciable in the design codes.

Tafheem *et. al.* (2013) modelled a six-storey steel building using ETABS 9.6.0 software and analysed it for lateral earthquake and wind loading, dead and live loading. Using concentric X brace and eccentric V brace having hollow steel sections (*HSS*). The performance of building was evaluated in terms of lateral storey displacement, storey drift as well as axial force and bending moment in columns at different storey levels and was compared with unbraced structure.

It was found that concentric X braces reduce more lateral displacement but attract large inertia force during earthquake also provide more stiffness and hence less ductility in comparison to eccentric type bracing. In eccentric type bracing, lateral stiffness of the system depends on flexural stiffness of beams due to eccentric connection of braces to beams. Although bracing decreases the bending moments and shear forces in columns but they increase axial compression in the columns to which they are connected. In comparison to unbraced structure influence of bracing system on inter-storey drift was found to reduce from bottom to top. In corner columns axial forces increase greatly from top to bottom. Maximum corner column moments were found to be very high at bottom two floors and drastically reduced up-to top floor in braced frame in comparison to the unbraced frame. Eccentric V- bracing was found to develop more column moments in comparison to Concentric X-bracing.

Choi *et. al.* (2017) reviewed the design requirements of throughout beams and columns of the braced frames considering 5 bays and storeys inverted V (chevron) braced OCBFs according to KBC2009. They investigated the collapse capacities, characteristic behaviour and collapse modes of the selected frames and suggested to use bigger sized column to prevent weak storeys at lower storeys in OCBFs.

Dilipkumar *et. al.* (2018) highlighted the advantages of chevron-based zipper braced frame over concentrically braced frames to be used in RCC structures. Some of the merits were, better ductility, strength, uniform inter-storey drift and control of vertical unbalanced forces in Chevron braced frames. Its design method was said to be conservative. The seismic strength of the brace intersected beams of inverted V-braced frames was found to be more than that of the two storey X-braced frames. Whereas zipper braced frame were not found to be economically efficient for low and mid-rise buildings.

Razak *et. al.* (2018): Compared various types of bracings (*including BRBs*) and found that amongst all conventional bracing types, eccentric braces reduced the potential damage to the structure and reduced the overall cost of the building. Safety of the occupant in the building can also be ensured by using eccentric braces, as they have been found to delay the structural response toward the earthquake.

1.4.3 Researches done on 'All-Steel BRBs'

Corte *et. al.* (2010, 2011): Reinforced concrete (RC) building were retrofitted using '*all-steel*' BRBs. Significant benefits were observed in the overall response of the system, in terms of stiffness, strength and energy dissipation capacity. But, the global out-plane deflection of BRB was observed, which would be considered as a detrimental effect (*the finalized configurations of the BRBs developed under the research work presented in this thesis didn't show any kind of significant out-plane displacement of overall BRB*).

They also reviewed the steel buckling-restrained braces and quoted that using flexible MRFs in combination with BRBFs can provide re-centring capability along with significant post-yielding stiffness.

Ma *et. al.* (2012): It was found that the low-yield strength steel experienced plasticity under small strain. This steel was expected to fabricate all-steel BRBs with better ductility and energy dissipation capacity. But, the hysteresis curve of BRBs clearly indicated that loss of strength at the later stages of loading; even less than 3% drift.

Hoveidae *et. al* (2012, 2015): According to them, key requirements of buckling restrained braces are the performance of BRB by avoiding overall buckling until the brace member reaches target displacement and sufficient ductility. Casing flexural stiffness could significantly affect the global buckling behavior of a brace, regardless of the size of the gap. Many cases of hysteresis curves for their developed BRBs showed serious strength degradation (*local buckling of core*). The analyses were conducted to examine the influence of the core and casing interface on buckling.

Piedrafita *et. al.* (2015): A new type of perforated core All-Steel BRB was introduced. Very good hysteretic behaviour was achieved. The mechanism of inelastic deformation of core was different from that of the conventional BRB and the other All-Steel BRBs.

Hosseinzadeh *et. al.* (2016): Seismic evaluation of all-steel buckling restrained braces was done using finite element analysis (*Some of the hysteresis curves showed strength degradation on the tension side. Such behaviour has not been observed generally in the BRBs and which would be detrimental*). The FEM based results revealed that the gap sizes of larger than 10 mm led to strength deterioration of the BRB in compression which could be attributed to the local buckling of the braces. Core section of specimens which possessed higher buckling capacities had higher mode buckling failure.

Judd *et. al.* (2016): Web-restrained brace (WRB) frame sub-assemblages using a built-up double web I-shaped steel section as the buckling restraining mechanism was analysed. The specimens were subjected to the quasi-static displacement protocol. Good hysteresis curves were obtained with no visible degradation up-to 2% story drift.

Bai *et. al.* (2016): They analysed a previously experimented BRB (*to be included in a RC moment frame specimen*) using FEM based numerical simulation software and were able to match the hysteresis curve very close to the experimental result.

Shen *et. al.* (2017): They developed an All-steel tube-in-tube buckling controlled brace (TinT-BCB) and its efficiency was ascertained experimentally and numerically using finite element simulations. TinT-BCB demonstrated controlled global and local buckling with stable and balaced cyclic response using buckling controller.

Mirtaheri *et. al.* (2018): They tried to improve the behavior of BRBs through obtaining optimum steel core length. It was proposed that the shortest length where failure does not occur is the best length of the core that can be proposed as the optimum steel core length of BRBs. i.e., suggested to use shorter core length.

1.4.4 Identification of research gap

As far as the steel structures are concerned, most of the researches on the non-ductile conventional braced frames (NCBFs) have been conducted to either study their structural behavior (*as clear from most of the literatures cited here*) or to develop new types of braces/ technologies like damper, zipper-brace, Y-brace or various kinds of BRBs (*most of the newly developed All-Steel-BRBs have been found to undergo global buckling*) etc., for new constructions. But, the upgrade of the existing braced frame structures for improved buckling load resistance (*or buckling behaviour*) has not been given the needed importance. A very few researches have been conducted in the past regarding the renovation of steel NCBFs and ordinary concentrically braced frames (OCBFs).

The available retrofitting related researches included filling of the hollow section with concrete, replacing connections (*required dismantling*), replacing beams with SCBF compliant beams, replacing braces with BRBs or completely changing the bracing arrangement (*like conversion of chevron bracing into two-story X-braced configuration*). These retrofitting measures required serious structural intervention and severely disrupted the occupants/users of those structures (Rai and Goel 2003). They were either very costly, complicated to execute. Some of them required pre-fabrication, sophisticated equipment, complete replacements of various components even if they were functional/ undeteriorated, dismantling of structure or change of arrangement of the members etc.

The requirement to devise least interventive, least disruptive, expeditious and handy yet effective retrofitting measures was felt here. Also, a basic prescription given in various researches and survey reports on old truss bridges was to avoid complete replacement of the old structures by retrofitting them as economically as possible, maintaining their historical appearance. This research work was intended to fulfill all these requirements.

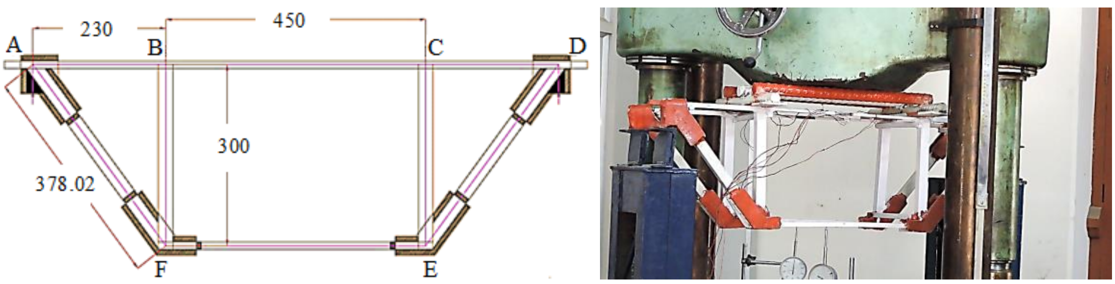
1.5 LAYOUT OF THE CHAPTERS FOLLOWING THE INTRODUCTION

1.5.1 Section 1: Linear Analysis of Selected Steel Structures

Experimental and numerical buckling analysis of space frame (Chapter 2)

Second chapter discusses about an experimental and numerical analysis done to better understand the buckling behaviour of the welded steel bars. Vaswani (1961) experimentally determined the buckling load and compared with the empirical results. When the critical load is reached, the symmetrical frame carrying symmetrical load buckles into an asymmetrical configuration (Lu 1963). Yura (1971) discussed the importance of exterior columns in stabilizing the inner columns in the same storey of an unbraced framed structure. It was concluded that columns in multi-storied buildings can often be designed on the basis of effective length equal to actual storey height.

Kishi *et. al.* (1997) approximately determined the column K-factor using inelastic column buckling. Chen (1999) introduced an end fixity factor by modelling the connection as a rotational spring. Gengshu and Zuyuan (2001) approximately determined the effective length of columns in a partially braced frame building by varying the brace stiffness based on the assumptions given by some previous authors. They found that the relation between the critical load and brace stiffness is linear. Kaveh *et. al.* (2006) calculated the buckling load of the frames using the effect of semi-rigidity of the joints given by Chen (1999).



a) b)

Figure 1.7 a) Geometry of the Specimen, b) Experimental Setup

IS 800 (2007) recommends the use of effective length ratio as 0.65 for rigid connection and also gives various equations based on stiffness (*that is basically dependent on the moment of inertias of the connected members*). Whereas, in the present work, experimentation (shown in Figure 1.7.b) and numerical simulation of rigidly connected welded bars in a frame showed that the effective length ratio for the buckling members (*'BC' shown in Figure 1.7.a*) not only depends on the moment of inertias but could vary for different cross-sections of buckling members and with the variation in the cross-sections of the connecting members also.

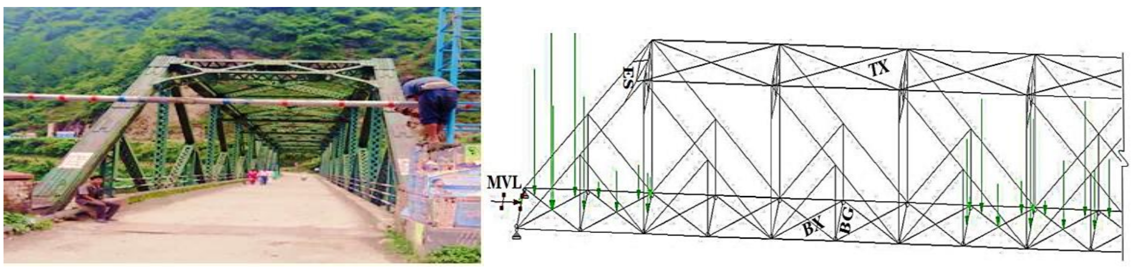
Generally, for analysing a rectangular bar/beam, buckling about the perpendicular cross-sectional axes (*X and Y*) have been considered, as it has also been observed in the FEM based numerical analysis using Abaqus CAE 6.14 (2014). The results and conclusion drawn out of this study would act a suggestive report for the drafts like BIS structural design draft (2015). Virgin (2018) showed that if the columns buckle toward each other resulting into the contact between them then the post-buckling stiffness will increase.

Webber *et. al.* (2015) made various improvements in method of calculating effective length, which was previously done by considering the isolated critical column of the frame forming an equivalent pinned Euler column (*which was generally utilized by codes of practices*). This work includes some works on the future work scopes enlisted by Webber, as it would incorporates the observations of the buckling modes, finding effective length through finding the point of contra-flexure for a considered 3D frame.

Promoting the loading class of the old steel truss bridges (Chapter 3)

Along with the development of new technologies and concepts of construction of the steel truss bridges, concerns about preserving and effectively using the historic bridges have been raised on various platforms. One such workshop named Historic Bridge Workshop was held at AASHTO headquarters, in Washington, D.C., December 3-4, 2003, where Historic Bridge Policy Statement was presented (DeLony and Klein, 2005). The SRI Foundation (2011) reported the bridges constructed between 1885 till 1955 in various states of USA that required to be rehabilitated for historical interest. Similarly, Thiel *et. al.* (2001), McKell *et. al.* (2006), and Connell *et. al.* (2010) also reported on the requirement of the rehabilitation of the truss bridges. In an investigation, Hatfield (2001) found that the modern specifications may allow larger stresses in the members of old bridges than did their designers. Mesler (2007) expressed his concerns for preserving these bridges as the unique examples of craftsmanship.

An old steel truss railway bridge tested by Blanksvärd *et. al.* (2014) that remained elastic up-to about 3 times the original designed load and failed in ductile manner. It showed the qualification of such bridges to serve for more years to come. The concern in most of the rehabilitation projects was to preserve the bridges by providing economical safety features that maintain their historic appearance and keep them functional.



a) b)

Figure 1.8 a) Picture of Bridge, b) moving vehicle load (shown by arrows) in STAAD model
Note: *BG, ES, BX, TX* and *MVL* represent bottom girder, end sway struts, bottom cross braces, top cross braces and moving vehicle load respectively.

Kim *et. al.* (2015) used overlapping steel sections for construction of a continuous truss having concrete filled in the bottom chords around the intermediate supports to overcome the negative moments. Huang *et. al.* (2018) used concrete filled tubes (CFT) for both the top and bottom chords and found that the increase in concrete compressive strength of bottom chord had negligible effect on the strength of the truss. Books by Victor (2004) for bridge engineering, Raju (2007) for the concepts of prestressing and Duggal (2010) for steel design and analysis, were also referred here.

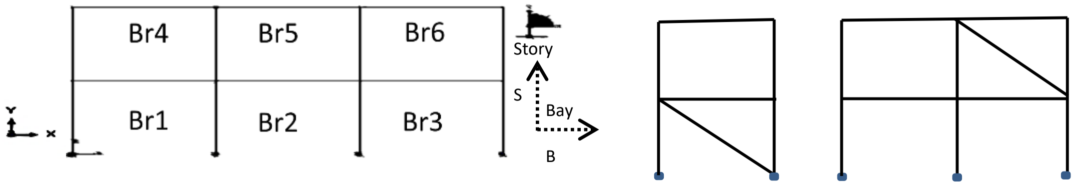
For the analysis of the bridge shown in Figure 1.8, STAAD Pro software (2015) was used because of its inbuilt feature for the provision of moving vehicle loads. Codal provisions were referred from the concerned Indian codes, IS-456 (2000), IS-800 (2007), IRC-24 (2010), IRC-6 (2010, 2016). Details of no-shrink micro-concrete were obtained from the manual given by FOSROC (2019).

Concrete filled (*main chords*) closed tubular section bridges have been the new type of bridges but the old bridges have not been retrofitted like them. It was tried here to utilize the available open sections of the members of the selected bridge for renovation. This has been done without much altering the existing state of the bridge and without hampering the aesthetics of the bridge. As the open section of the main chord members of the selected bridge provided an ease of introduction of modifications. Two different techniques suitable as per the behavior of the stresses were introduced in the analysis for the renovation of the selected bridge.

Linear buckling analysis of braced frames under axial and lateral loadings: The effect of bracing location (Chapter 4)

Under compressive loading, the slender members are prone to buckling. As long as the load is small, axial shortening of the member is observed and once the certain critical load (*Pcr*) is crossed, a sudden out-sideways bowing of the member occurs giving rise to large deformation. The effect of bracing location on the buckling behavior of steel braced frames has been studied in this part. The behavior was examined in terms of critical load

(P_{cr}) values obtained from numerical analysis using linear perturbation buckling analysis method which was based on Eigen value problem. Nomenclature and geometry of the considered steel frames has been shown in Figure 1.9.



a) b) c)

Figure 1.9 a) Numbering of braces (*in Abaqus Model*); b) One bay - two stories frame (B1S2) with brace in Br1 location; c) Two bay - two stories frame (B2S2) with brace in Br3 location

Abaqus CAE software (2014) has been used to model the frames and to obtain the 'P_{cr}' values. The frames were subjected to compressive loads in vertical and horizontal (*lateral loading*) direction separately. To validate the method of numerical analysis, a bar with various end restraints was cross-verified with classical Euler critical buckling load results. The results were found to be very much in acceptance with classical method results.

Inclusion of lintels or lintel bands in chevron braced frames (Chapter 5)

Lintels have been used basically to bear the wall load above the openings and to transfer the load in infill walls of framed structures and in the load bearing walls of the masonry structures. Lintel bands have been used generally in the form of steel straps around the masonry buildings or as the lintels extended to the edges to the load bearing walls or between columns of RCC framed structures to improve lateral load capacity induced of the seismic forces (Murthy *et. al.* 2004, Chaudhary *et. al.* 2015 and Dutta *et. al.* 2016).

In this analytical study, the effect on the buckling behaviour of steel frames on inclusion of lintel bands (*as struts between columns*) and the lintels (*as struts between the eccentric/ concentric chevron braces*) has been studied and compared with other type of bracing.

Sarno *et. al.* (2008) designed a 9-storey moment resisting steel frame (MRF) and then retrofitted it using various bracing configurations and improved its performance under earthquake ground motions. Borri *et. al.* (2009) unlike generally used used composite reinforced masonry ring beams in masonry structure to improve lateral load resistance under seismic forces. Tasnimi *et. al.* (2011) investigated the in-plane seismic behaviour of steel frames with brick masonry infill having central openings in it. Formation of cracks along the strong spandrel beam resulting in higher frictional hysteretic damping was found to be partially depended on the lintel beam. In the numerical analysis of progressive collapse situation (*removal of an internal column*), Kim *et. al.* (2011) found that the braced frames worked better than the moment resisting frames.

Lignos *et. al.* (2011) conducted loading experiments for the prediction of side-sway collapse of steel moment frames and found that the commonly used symmetric cyclic loading histories provide insufficient information for modelling deterioration near collapse. Eghtesadi *et. al*. (2011) found that X-braces have only one lateral force resisting member in each span, so require stronger beams and columns. Decanini *et. al.* (2012) found that the provisions of strengthening measures like lintel bands and steel reinforcements around the openings of the infill panels checks the reduction of stiffness and strength under seismic loading. Gerasimidis (2014) generated progressive-collapse mechanism by removing the key element of the structure. It was found that, when the columns at the bottom stories are removed, buckling of column governs the collapse mechanism and when the removal of column happens at the upper stories, collapse mechanism is governed by the flexural failure of beams. Najarkolaie *et. al.* (2017) did experimental and analytical study of a single storied masonry infilled steel framed structure to investigate the effect of distributed and concentrated lateral loading and found that the formulas, mostly based on specimens with concentrated loading, are conservative.

The lintel bands can be included in the new construction of chevron braced frames and can also be used to upgrade the existing chevron braced frames. The horizontal lintel bands and the conventional steel brace have been used in earthquake resistant structures to improve the seismic performance of the structure and resist sway. In this numerical study, steel lintel bands were examined for their capability to resist buckling in braced steel frames in combination with chevron braces. The considered specimens have been shown in Figure 1.10.

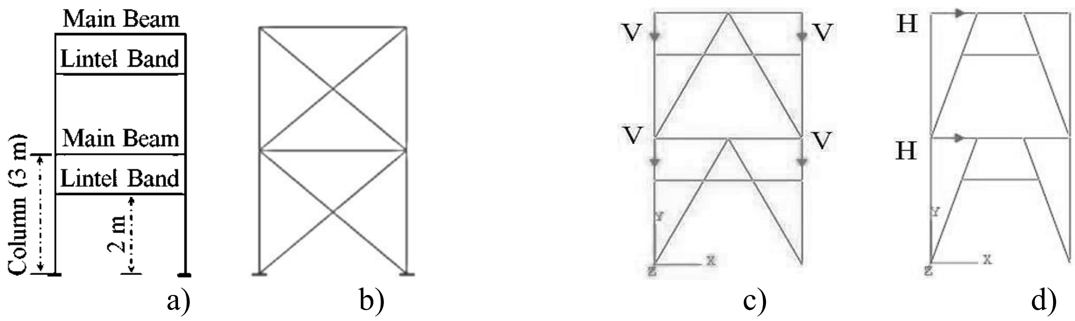


Figure 1.10 a) One bay two storied frame with lintel bands, b) X bracing, c) Vertical loading (*V*) in chevron braced frame with lintel bands and d) Lateral loading (*H*) in eccentric chevron braced frame with lintel

1.5.2 Section 2: Non-Linear Analysis of Older Braced Frame

Use of vertical and diagonal members to upgrade NCBFs (Chapter 6)

Use of vertical and diagonal members to upgrade the eccentric NCBFs

For many years, the braced frames have been the most common type of the primary seismic force resisting systems. Two major divisions of braced frames are concentrically or eccentrically braced frames. Concentrically braced frames proved to be economical option over the moment resisting frames under moderate seismic loads.

The severe seismic loads could result into severe loads reversals and unreliable structural behaviour of CBFs. Eccentrically braced frames have been found to produce a better hysteretic behaviour and less uncertain structural behaviour at severe seismic loads.

Eccentrically braced frames have been broadly divided into three types as short, long and intermediate based on their structural behaviour. It has been suggested to provide small eccentricity (*short links*) for higher stiffness (Popov 1983). Foutch *et. al.* 1987 and Roeder 1989 found that the beam panels with little eccentricity in brace connections (*beam-brace splice panel zone*) acted as very short links for energy dissipation under low and moderate seismic loads but these links weren't effective for severe seismic.

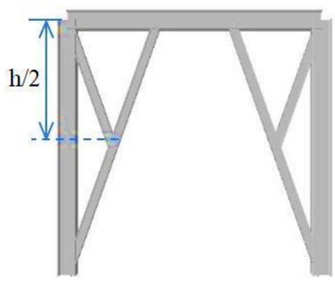


Figure 1.11 Modified EBF with additional diagonal at central height (*h/2*)

Before the introduction of detailed seismic design provisions for eccentric braces, many of such configurations were just for the architectural purpose and to accommodate architectural requirements for openings. Such eccentrically braced frames were the object of the introduction of retrofitting by adding diagonal members (shown in Figure 1.11).

Use of vertical and diagonal members to upgrade the concentric NCBFs

As far as steel structures are concerned, most of them are generally equipped with addition members for the lateral load resistance and for the seismic energy dissipation that are called as primary seismic force resisting systems. These primary seismic force resisting systems include various types systems but the most common type is the braced frames. When considering old concentric chevron braced frames an unbalanced force resulting from braces acts on the beam, because of which strong beam has been suggested in the SCBF provisions (ASCE 2016). But in the old chevron braced frames this problem still exists (Sen *et. al*. 2014).

a) Braced frame with added diagonal b) Braced frame with added vertical

Figure 1.12 Braced frame having added diagonals and verticals

Some researchers found the old braced frames to be capable of resisting moderate seismic forces and found braces or the weak beams as secondary members for retrofitting (Sen *et. al*. 2014 and Sizemore *et. al*. 2017) and some found it necessary to replace weak beams with SCBF based beams (Rai and Goel 2003). Tsuji (1988) experimentally showed the drawbacks of using weak beam in chevron braced steel frames. The deforming nature of weak-beams also had an adverse effect on the columns and the connections (Robert and Trembley, 2000, Sabelli, 2001).

Numerical analysis has been considered as one of tools for understanding the behaviour of steel braced frames. It is difficult to access the manufacturing defects and imperfections, locally concentrated errors in the actual structures as they vary for every experiment and for every loading setup conducted in various locations throughout the world. Even-though the local behaviour of the braced frame was not captured in the numerical model by Sen *et. al*. (2014) and Sizemore *et. al*. (2017) but the global behaviour matched with the experimental one. Narayan *et. al*. (2020) found that the Abaqus software (2014) simulated the welded frame behaviour very much in competence with the experimented one. The effective length of the buckling members was closely achieved and the size of the connected member also affected the effective length.

This study provided a simple and effective means to overcome some of the major deficiencies of the chevron braced frames without causing extensive structural intervention or disruption to the occupants. This has been done without replacing any member and without increasing the size of the brace.

Decreasing the slenderness ratio of the braces below 30 resulted in a sudden decrement of strength of the overall frame at the later stage of the cyclic loading (Wakabayashi *et al.*, 1977). In support of the same, Narayan and Pathak (2020) found that when the slenderness of the braces was close to the slenderness of the beams and columns, it resulted into strength loss and adverse effects on the columns.

In this work, to overcome some of the major deficiencies of the chevron braced frames by avoiding the problems occurred due to the increment in the brace size, four different bracing configurations of chevron braced frame obtained after including additional diagonal and vertical bracing members were numerically analysed. Three configurations had additional diagonal members connected to the existing brace at the various locations and one configuration had additional vertical members (as shown in Figure 1.12).

Transforming chevron brace to multi-level eccentric-chevron (Chapter 7)

One of the main objectives of seismic design of braced frames is to consider their restoring force characteristics. To achieve the desired drift limitation in MRFs, under moderate earthquakes, use of heavy and deep beams proved to be very costly; whereas, the braced frames proved to be economical. Under the extreme lateral loads, various bracing configurations pose various challenges and problems. Testing a brace (as strut) individually under cyclic axial loads shows serious degradation of compression strength during reloading after buckling. Fortunately, such poor performance of individual brace doesn’t decrease the capacity of the multiply redundant frames to the same extent.

Secondary action was found to resist 20% of the lateral shear forces in the initial loading stages. Beyond the initiation of failure in the primary SFRS (*seismic force resisting system*), 60% and 50% of the lateral shear was resisted by the secondary actions in study by Foutch *et al*. (1987) and Fukuta *et al*. (1989) respectively.

Full testing setup (*traditional beams and columns with section comparable to that of the braces*) used by *Wakabayashi et. al.* (1980) was as shown in Figure 1.13 (a). Experimental inputs, member sections and properties obtained in that report were utilized in present study for the selection of generalised chevron NCBFs as shown in Figure 1.14 (a), (b).

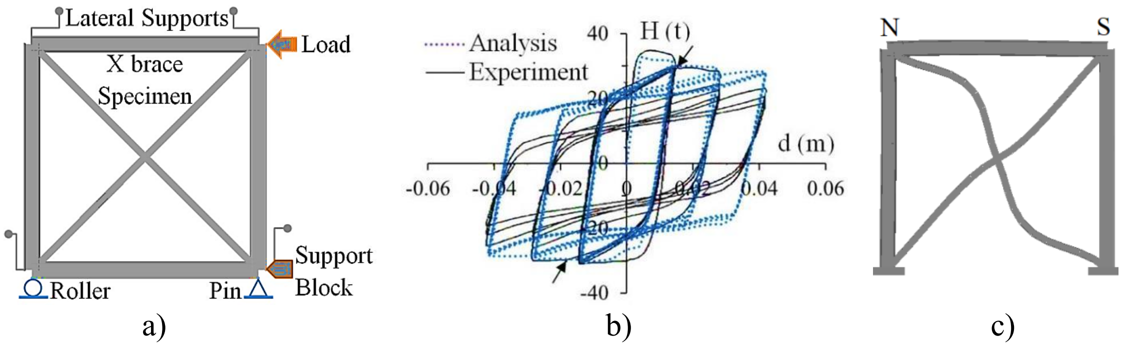
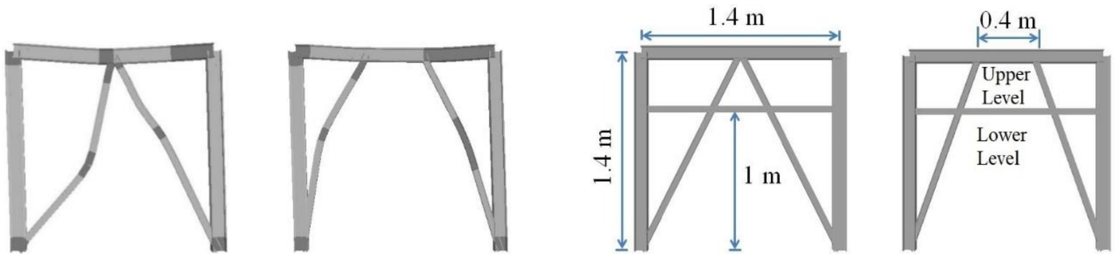


Figure 1.13 a) Experimental loading setup b) Numerical and experimental hysteresis loops c) Buckling analysis of X-braced portal frame

In a study of three story SCBF frames (*having WF braces*) by Lumpkin *et al*. (2009), the global drift reached up-to 5%. In a study of eccentrically braced frames by Popov (1987) no deterioration was observed in the braced frames up-to 1.5% drift and stable hysteretic behaviour was observed up-to very high drift ratios. In the numerical analysis of three-story chevron braced frames by Sabelli (2001), maximum story drift of 3.9% was observed. In the referred report, Wakabayashi *et al*. (1980) used 4% drift limit for both braced (*X-braced frame couldn't sustain above 3%*) and un-braced frames. In the present study, drift limit for one-story and drift range for two-story chevron CBFs was 3%; whereas, drift limit for one-story and drift range for two-story chevron EBFs was 4%.



a) *Ch* b) *Che* c) *ChLN* d) *CheLN*

Figure 1.14 a) and b) show deformed concentric and eccentric chevron *NCBFs* respectively; c) and d) respectively show concentric and eccentric chevron braced frame having lintel bands

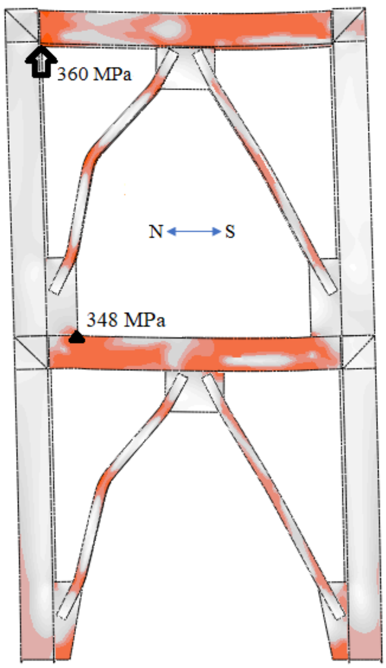
Here, the numerical analysis was done using Abaqus software (2014) to simulate the behaviour of the braced frames tested by Wakabayashi *et al*. (1980) and it was able to do so up-to a very good degree as clear from the hysteresis loops shown in Figure 1.13 (b). Even the critical points (*shown by arrows*) observed in the experiment matched with the numerical analysis results. For the in-plane buckling of X-braces, hysteresis loops in both partial system and full system test setups, were similar and similar results were obtained here, for the X-braced portal frame. The buckling mode obtained by linear-perturbation buckling analysis of the X-braced frame showed negligible deformation in frame members (shown in Figure 1.13.c).

In tested X-braced frame, after 8th cycle, serious cracks developed in the specimen and for the continuation of loading after this stage; because of which little higher strength was obtained in the numerical analysis results in comparison to experimental results. The failure progress along damage continuum is hard to capture and over that it is very difficult to predict such damage locations.

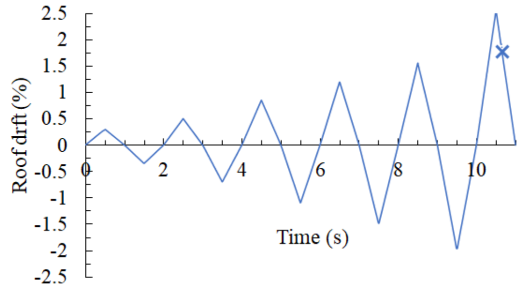
Murthy *et al*. (2004) used twin lintel-belt and Choudhury *et al*. (2015) used steel band, to retrofit the masonry buildings. Dutta et al (2016) suggested retrofitting in a RCC framed structures by filling empty corner bays with brick masonry and by providing lintel beams in open bays to avoid soft story of bottom storey. A method to upgrade the existing chevron CBF/EBFs using lintel bands has been discussed here. The resulting arrangement was a multi-level eccentric chevron (MLEC) brace generated out of the combination of a chevron brace and a lintel band, as shown in Figure 1.14 (c), (d). It incorporated the advantage of the frame action and multiple redundancies of the frames for providing both strength and ductility without changing the beams or braces with heavier ones and without disrupting the existing state of the chevron braced frame.

Conversion of non-seismic chevron brace into stiffened-casing dual-sleeve buckling restrained brace (SCDS-BRB) (Chapter 8)

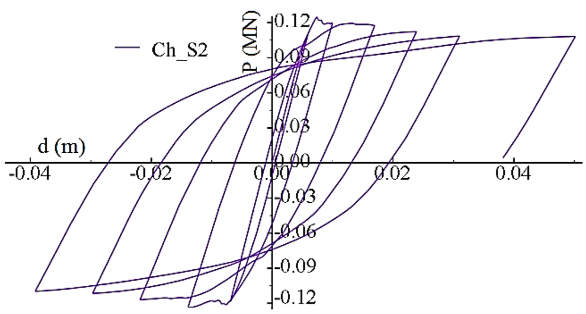
The reference Chevron NCBF was modelled and analyzed by adapting the member specifications given in an old experimental report by Wakabayashi and Tsuji (1967). The deformed shape of the reference NCBF and the hysteresis curves matched closely with the experimental results to a very good extent. The proposed renovation method was implemented in the NCBF by modelling and conducting the numerical analysis.



a) Deformed shape (*Ch_S2*)



b) Lateral displacement loading protocol

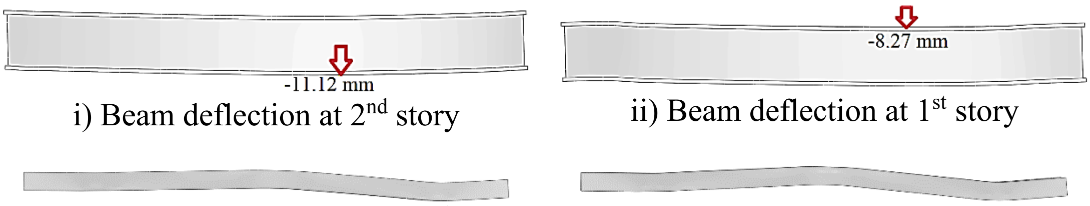


c) Hysteresis curve

Figure 1.15 Analyzed two-story Concentric Chevron braced frame (*Ch_S2*)

The deformed two-story Chevron braced specimen (*Ch_S2*), loading protocol and the resultant hysteresis curve has been shown in Figure 1.15. The darker the shades, higher the stresses in that region of the deformed specimen (Figure 1.15 (a)). It can be seen that the columns were not stressed much (*except at the base*). Maximum stress was observed in the top story, at the bottom of the north side beam-column connection (*360 MPa, indicated by arrow*). Maximum stress at the brace-column connection was observed at the north end (*348 MPa, indicated by triangular dot*) above the first story beam.

The maximum stress and displacements were observed at the peak of the last half-cycle loading. The loading was continued till the point marked by 'x' sign indicated in Figure 1.15 (b). The sudden peak in the horizontal reaction (*P*) with respect to the applied lateral displacement (*d*) in the initial loading cycles (*buckling of the braces*) resulted in the strength degradation at the later stages of loading (*as observed in Figure 1.15 (c)*).



i) Beam deflection at 2nd story

ii) Beam deflection at 1st story

iii) Ineffective tension brace at 2nd story

iv) Ineffective tension brace at 1st story

Figure 1.16 Final state of deformation of beams and braces

It can be seen from Figure 1.15 (a) and Figure 1.16, that after the buckling of north side braces, the braces expected to undergo tension could not undergo tension (*ineffective; remained in compression*) due to the significant downward deflection of the beam. Methodologies for the upgrade of the concentric and eccentric Chevron braced frames have also been suggested by Narayan and Pathak (2021, 2022), which would cause minimal structural intervention and least disruption to the occupants without the replacement of any member. But, there also, the hysteretic behavior of the concentric Chevron braces showed considerable strength degradation.

The objective of the work presented in this section was to improve the hysteresis behavior of the concentric Chevron NCBF. To achieve it, an arrangement for the conversion of the 'existing' braces into BRBs was devised after a huge number of trails. Various new types of all-steel BRBs have been developed by many researchers (Piedrafita, *et. al.* 2015; Judd *et. al.* 2016; Seker and Shen 2017). The BRB developed in present study was also an all-steel BRB. It resulted in a configuration where horizontal stiffeners were added to the buckling restraining casing and to hold the long-grooved end of the casing, secondary sleeves were provided in the non-yielding upper portion of the brace.

1.5.3 Original Contributions (Chapter 9)

"Original Contributions" chapter gives the discrete details about the novelty of the research work. In this chapter, the specific contributions made to either theory, design procedure, or the introduction of an assessment process by each work conducted here will be discussed. To discuss the novelty of the work, the focus has been given to the scientific and fundamental originality of this research work.

1.5.4 Summary and Conclusions (Chapter 10)

"Summary and Conclusions" chapter includes an elaborate description of the conclusions drawn from the main text chapters. Other than that, each of the main text chapter has been accompanied by the brief "concluding remarks" given at the end of those chapters. At the end of this chapter, a section called "scope for further works" has been included for the prospects and opportunities for the further research works that can be pursued or executed as an extension of the work presented here.

SECTION 1: LINEAR ANALYSIS OF STEEL STRUCTURES

In this section linear analysis methods have been implemented to carryout various studies on various braced frames. Firstly, a buckling experiment was conducted on an under-slung bridge type specimen. Utilizing the data obtained from the experimental analysis, linear perturbation buckling analysis was conducted in a finite element-based software (Abaqus CAE) and various extrapolations were made. It was conducted to get an insight about the buckling behaviour (*buckling modes and effective length ratio*) of the frame members. Next, an old and historic bridge was analysed in STAAD Pro software where its existing state was securitized in detail. Here, along with the conventional truss analysis, the combined axial and bending moment analysis was also performed. A renovation strategy to upgrade the bridge from the existing state of the lowest vehicular movement class, the class-B loading to highest 70R loading class was proposed.

In the next two chapters, linear perturbation buckling analysis was conducted on the selected braced frames considered them to be the non-ductile braced frames (NCBFs). First analysis was done to ascertain the effect of the location of the braces in resisting the lateral loads and second analysis was done to suggest the measures to strengthen the existing braced frames so as to improved their lateral load resistance.

The linear perturbation buckling analysis would be incapable of analysing the braced frames under repeated/ cyclic lateral loading, analysing them for inelastic deformation and incorporating the contact interaction based internal modifications in the sections of the members of the braced frames. So, in Section 2 of this thesis, non-linear analyses were conducted to ascertain the improvement in the ductility of the braced frames on incorporating the proposed renovation strategies.