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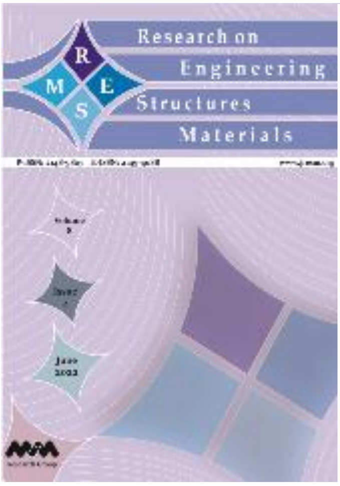

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Research Article

Bi-diaphragm elastoplastic haunch retrofit solution for ill-detailed RC beam-column joints

Abhay Gujar*[a], Sachin Pore[b]

Department of Civil Engineering, DBATU, Lonere, Maharashtra, India

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Abstract

Failure of ill-detailed connections is a primary cause of catastrophic, cascading failure of Reinforced cement concrete (RC) frame subjected to seismic loading. A performance based, 3 dimensional (3D), Bi-Diaphragm bolted metallic haunch retrofit solution is proposed, to avoid brittle shear damage to the connection panel zone. Welding is avoided in fabrication of the proposed device to avoid brittle failure in any part of the assembly. The haunch is designed to alter the strength hierarchy of connection subassembly, ensuring plastic hinge formation in beam before any other type of failure occurs. Assessments reveal that, 3D geometry improves torsional stiffness (+277%) and resilience (+94%) over those of an equivalent planer haunch. Numerical analysis is done to estimate the stiffness of haunch. A flow-chart illustrating procedure to design the proposed haunch is presented. The efficacy of proposed solution is evaluated by performance comparison of a Parking+6 storey frame, with and without retrofit solution, subjected to push-over (+87% rise in lateral load at yield, resilience increased by +37 %) and non-linear seismic analysis (Max. roof displacement reduced by -106% and storey drift by -154%). Masonry walls in habitable floors are modeled as equivalent diagonal struts to replicate much essential soft-storey effect. This study is focused on retrofitting exterior beam-column connections owing to their vulnerability reported in the literature. Analytical and numerical assessments confirm the efficacy of proposed solution in mitigating failure during seismic excitations.

1. Introduction

Experiments [1-2]and forensics [3] of earthquake-damaged structures have exposed inherent deficiencies in buildings designed and detailed for gravity-only loads. The absence of capacity design principles and inadequate ductile detailing are responsible for failure of such structures [4]. Hakuto et al. [2] conducted a series of investigations, pertaining to sub-standard column-beam connections. For a few of the specimens, the longitudinal bars of beams were hooked away from joint core. Remaining specimens had bars hooked inside the joint panel. Subassembly with bars hooked inside connection panel zone appeared to be more efficient in resisting joint shear owing to development of diagonal compressive concrete strut mechanism and reinforcement truss action. Reinforcement truss action is composed of bond between concrete and steel along with tensile strength of rebar. Vulnerability of joint core zone for deficient connections was highlighted by experiments [3] on scaled down (2/3) sub-standard joints. Pampanin et al. [5] proposed that principal stresses are a better measure of connection performance. A value of principal stress (Pt), giving rise to 1st shear crack in connection, can be assumed as $Pt = 0.2\sqrt{fck}$.

Column-beam joint is found to be the weakest link causing a progressive collapse of the building subjected to lateral loading. Efficient strengthening or retrofitting solution,

*Corresponding author: abhay.gujar@dbatu.ac.in
[a] orcid.org/0000-0003-3294-3321; [b] orcid.org/0000-0001-8924-3823
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generating required protection to joint panel zone, while altering hierarchy of strength between different components of subassembly, is required to enhance the performance of building frame during seismic activity. Accounting for stiffness of infill brick masonry on upper floors is essential while performing seismic analysis of buildings with parking story. Sudden change in stiffness above parking floor tends to develop a soft story mechanism.

Improvement in performance of ill-detailed connections can be achieved through various active and passive retrofitting techniques. Ghobarah et al. [6] employed corrugated sheets for joint wrapping, carbon Fiber Reinforcement Polymers (CFRP) was used by [4], diagonally welded singular plate haunch by [7]. Jayasooriya et al. [8-9] employed diagonal haunches in the form of buckling resistant braces. Geo-Polymer [10] wrapping was also used to enhance the performance of the substandard connections.

Several investigations using different configurations of 'X','K' and, 'V' braces, were carried out by researchers in order to improvise lateral load resilience of RC frames, with sub-standard joints. It was observed during an analytical investigation [11] that, use of rigid, non-buckling diagonal braces tends to improvise plastic zone behavior of frames with weak column-strong beam type of configurations. Experimental investigations [12] revealed that use of X-braces as diagonal stiffeners improvises shear-capacity of structures, in the plane of braces. Another study [13] noted that X type diagonal braces are more efficient in case of shallow buildings, rather than tall buildings. Experiments [14] also revealed that ultimate load carrying capacity (LCC) and initial stiffness is considerably enhanced due to use of eccentric and concentric 'X' braces and with 'V' or reversed 'V' braces. Hu et al. [15] conducted investigations on 'X' type braces. It was observed that rather than increasing number of columns, provision of 'X' braces efficiently improvises the performance of the building. Another Investigation on 'X' type braces [16] claimed that transverse load carrying capacity of deficient frames increases three times due to the proposed bracing system. Rahimi et al. [17] carried our numerical assessment of 'X' type braces to conclude that 'X' braces decrease the lateral drift of the frame and the shear demand on the connections. Jafari et al. [18] studied non-linear performance of RC frames retrofitted by diagonal knee braces. It was observed that, knee braces can increase column axial load by 15% for a G+2 building and by about 7% for a moderately tall (8-12 storey) building.

Dynamic time-history analysis [19] was done on 5 storey and 7 story RC frames retrofitted using eccentric steel-braces. Significant reduction in storey drifts as compared to as built frames was observed. In past, a number of researchers Rahai et al. [20]; Ramin et al. [21]; Naghavi et al. [22]; Ozcelik and Erdil [23]; Qian et al. [14]; Sutcu [24]; Godinez et al. [25]; Fateh and Hejazi [26]; Du et al. [27]; Kaviani et al. [28] investigated efficacy of various types of 'X' type, diagonal concentric or eccentric braces. The practical difficulty while using diagonal braces, across the bay, is that they pose a hinderance to an unobstructed passage within the habitable floor space. A better solution to cater this problem would be to use a knee brace type retrofit haunch element adjacent to beam-column connections. If the geometric dimensions of haunch element are designed keeping in view the headroom requirements, it will not cause any hinderance and will facilitate unobstructed use of the floor space.

Uang and Lee [29] were the first to propose a haunch retrofit solution for repairing and strengthening steel-moment-resistant frames damaged during Northridge-California (1994) earthquakes. Further investigations by Chen [30] established the utility of a haunched retrofit solution. Further study [30] of different haunch configurations yielded some conclusions, such as

- Haunch with stiffness below a lower bound, cannot arrest damage in joint panel zone.

- Column-beam connections have limited elastic deformation capacity thus, plastic deformation of haunches does not ensure enhanced resilience of the sub-system
- Having a haunch design, ensuring elastic behavior during entire loading range ensures best performance of the subassembly.

Haunch retrofit solution was improvised by Pampanin et al [31] . G. Genesio [7] devised a diagonal singular plate welded haunch, which was attached to beam column subassembly, through post installed anchor fasteners. The Bi-Diaphragm Haunch (BDH) retrofit solution proposed here is a modified bolted version of retrofit solution proposed by Genesio [7].

The practicability of any retrofit solution depends on factors like cost, reliability, replicability, invasiveness during installation, etc. This work presents the efficacy of a cost-effective, less invasive, 3-D bolted haunch, which can be introduced during or after the construction of a building. The primary concept is based on capacity design principles, wherein the stress path is deviated from passing through the weak connection zone. The path is altered in such a way that, a plastic hinge is formed in beam location, away from column face. Analytical and numerical investigations show that the haunch retrofit strategy increases strength and ductility of subassembly, and thus improves the over-all seismic performance of the building [30, 7].

2. Need for Proposed Retrofitting Solution

Typical deficiencies as reported in the literature [31-32] for gravity-only designed and ill-detailed connections are summarized as follows,

- Insufficient anchorage length in connection core for longitudinal reinforcement.
- Inadequate end hook detailing for transverse reinforcement.
- Inadequate amount of longitudinal and transverse reinforcement at connection core.
- Insufficient confinement in connection panel zone.

These deficiencies lead to catastrophic pancake failure of buildings subjected to seismic loading (Fig.1). To mitigate such failure, it is imperative to devise a solution, that can shift the critical stress path away from the beam-column joints, while ensuring failure governed by plastic hinge formation in the beam, away from face of the connection. Planer welded haunches [7] do not offer adequate post-buckling resilience. To avoid buckling, single plate haunch needs to be substantially thick. More-over, welded connections are prone to brittle failure. Other passive techniques like wrapping or increasing the strength of concrete in the connection zone by some means, do not remove connection zone from the critical stress path. The proposed BDH is expected to impart better resilience to the structure, while shifting critical stress path away from the connection panel zone. Further, literature study reveals that most of the performance evaluation done, does not consider the stiffness of infill brick masonry walls [31, 33] on the storeys above parking floor. Inclusion of brick masonry to structural stiffness explicitly shows vulnerability of parking floor joints and underlines the need for strengthening the same. In this work contribution of infill brick masonry to stiffness of upper floors is considered. Unlike planer haunches [7], proposed BDH is expected to impart a reasonable degree of lateral as well as torsional resilience to the subassembly. Although this solution seems to provide a practical cost-effective retrofit strategy, a systematic design procedure and numerical validation of the proposed technique is essential.

3. Performance of Ill-Detailed Connections

A series of tests [1, 2, 34, 35, 36, 37] highlight vulnerability of joint panel zone during seismic activity. It is reported that, absence of capacity design and insufficient quantity of confining and longitudinal reinforcement along with inadequate anchorage detailing are the primary causes of this vulnerability.

Fig. 1 Pancake failure Islamabad 2005 earthquake (Seismic design characterization of RC special moment resisting frames in Pakistan-field survey to laboratory experiments.)

In such cases a brittle shear failure is observed in exterior beam-column joints before occurrence of flexural hinges in the beam or column. The mode of failure changes depending on the location and detailing of the joint. In absence of confining reinforcement performance solely depends on concrete compression strut. Thus, for an exterior joint, strength degrades rapidly after the formation of first crack. Strain hardening can only be expected for interior joints [30]. Further discussion is especially pertaining to strengthening of exterior beam-column connections.

4. Effect of Haunch on Beam-Column Subassembly

A free body diagram of exterior column-beam subassembly assuming points for contra-flexure at mid spans of beam and column is illustrated in Fig. 2. Introduction of a haunch re-routs stress flow around column-beam subassembly as shown in Fig. 2. Moments and shear forces around the joint panel zone are significantly reduced. The critical moment in beam is shifted away from connection. This relocation of beam maximum moment can be exploited to ensure formation of a plastic hinge in beam before occurrence of any other failure mechanism. The efficacy of the proposed solution depends on X, α (Fig.6) and axial stiffness (K_d) of haunch [30]. The provision of haunch at top and bottom of beam eliminates axial force generation in the beam.

If the shear force in beam due to haunches is denoted by βV_b then BMD and SFD in terms of β can be drawn as shown in Fig. 3. Reduced beam moment at the face of connection is given as [30]

$$M_b = M_{bmax}\left[1 - \frac{\beta d_b}{2L\,Tan\,\alpha} - \frac{(1-\beta)X}{L}\right] \quad \text{where, } L = \left(\frac{L_{Bc}}{2}\right) - X \tag{1}$$

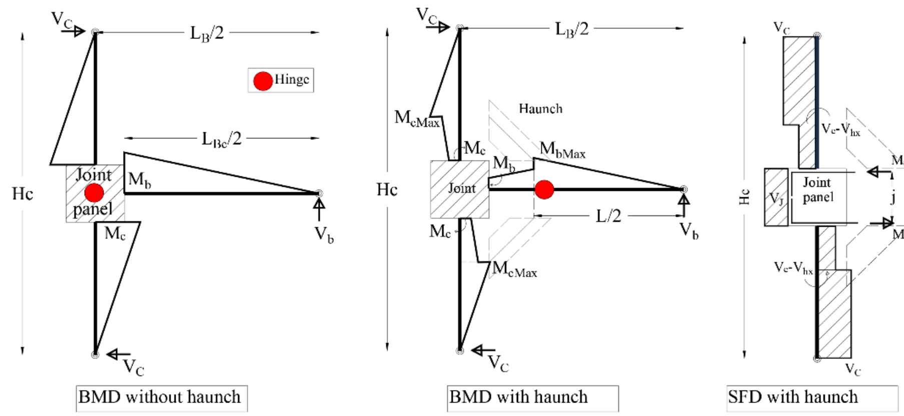

Fig. 1 Effect of haunch on SFD and BMD (Gujar & Pore, 2023)

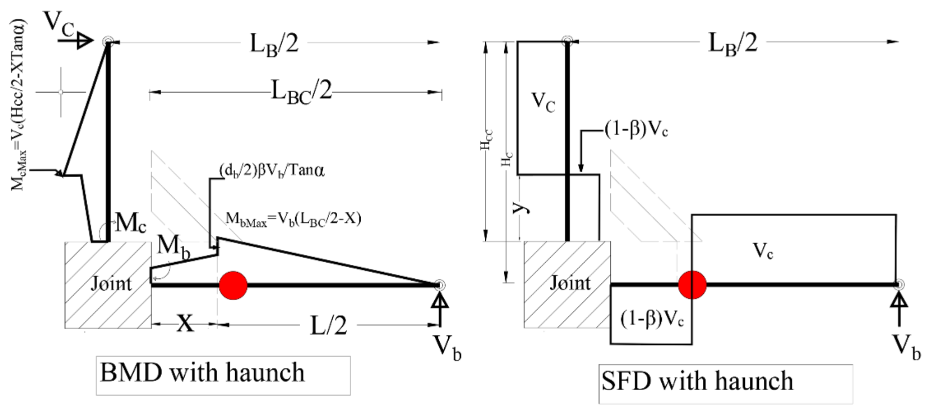

Fig. 3 BMD and SFD of haunched joint as a function of β

Reduced moment in column at connection interface is given as,

$$M_c = M_{cmax}\left(\frac{1-\beta_1 d_c Tan\,\alpha}{2H_1} + \frac{(1-\beta_1)\,X\,Tan\alpha}{H_1}\right) \tag{2}$$

Where

$$H_1 = \frac{H_{cc}}{2} - X\,Tan\alpha$$

and

$$\beta_1 = \beta\left(\frac{H_c}{L_B Tan\alpha}\right)$$

Maximum moment in column (M_{Cmax}) corresponding to maximum moment in beam (M_{bmax}) is given as [30],

$$M_{Cmax} = M_{bmax}\left(\left(\frac{H_C}{2} - \frac{d_b}{2} - X.Tan\alpha\right)\left(1 + {d_c}/{L_{Bc}}\right)\Big/ H_C\right) \tag{3}$$

Referring to moment diagram in Fig. 3 for details of symbols in the above expression, value of β >1 will yield negative shear force. Negative shear force will be more desirable to protect beam column joint in a better way [30]. β value is calculated from deformation compatibilities, between deformations of column and beam at the location of haunch and axial deformation of haunch. When flexural deformation of beam and column is also accounted for, then β is given as [30],

$$\beta = \left(\frac{y}{x}\right).\frac{6Ld_b + 3X.d_b + 6y.L + 4XY + \frac{2I_bL_by^3}{I_cXH_C} + \frac{3I_bL_by^2}{I_cXH_C} + \frac{3I_bd_cL_by^3}{I_cX^2H_C} + \frac{3I_bd_cHL_by^2}{I_cX^2H_C}}{3d_b^2 + 6yd_b + 4y^2 + \frac{12EI_b}{2K_{dav}X.cos^2\alpha} + \frac{6I_by^2}{X^2A_c} + \frac{2I_by^3}{XI_c} + \frac{3I_bd_cy^2}{I_cX^2} + \frac{3I_bd_c^2y^3}{2I_cX^3}} \tag{4}$$

$$L = L_{Bc} - 2X, \quad H = H_{CC} - 2y \text{ and } H_{Cc} = H_c - d_b$$

(Note: If only one haunch is used, $2K_{dav}$ is replaced by K_{dav})

Where, I_b and I_c are major moment of inertias of beam and column respectively.

For details of X, Y, α please refer Fig. 6 L_b is c/c span of beam d_c and d_b are effective depths of column and beams.

K_{dav} is average elastic stiffness of haunch. E is modulus of elasticity of concrete.

5. Proposed Retrofit Strategy

The primary concept is based on altering strength hierarchy by relocating plastic hinge in beam through axial straining of haunch element. Haunch is designed in such a way as to reduce moment acting on joint thereby protecting it from brittle shear failure. Principal stresses are found to be more reliable measure [3, 40] of predicting response of any element subjected to multidirectional loads, hence performance evaluation in this work is based on state of principal stress pertaining to an element. Elasto-plastic performance of haunch is so designed that while dissipating the energy, stresses in connection zone remain well below critical principal stress levels. It is further ensured that moment at connection does not exceed its moment capacity before beam hinging takes place. In short strength hierarchy of subassembly is altered to ensure,

$$V_{beamHinge} < \gamma 1.V_{columnHinge} < \gamma 2.V_{joint} < \gamma 3.V_{beamShear} < \gamma 4.V_{columnShear}$$

$\gamma 1,2,3,4$ $are\ factor\ of\ safety\ assumed\ in\ consistancy\ with\ code\ provisions$

(May be assumed between 0.7 to 0.85) [30]

6. Design Steps for BDH Element

6.1 Limit State of Column Hinging

It is imperative to check that column hinging does not take place prior to beam hinging. Haunch design should be such that beam hinging occurs prior to maximum column yielding capacity moment capacity Mpc is reached. Thus, first threshold is limit of column story shear given as,

$$V_{capacit\ column\ hinge} = \frac{M_{pc}}{\frac{H_c}{2} - \frac{d_b}{2} - X.tan \propto} \tag{5}$$

Mpc is plastic moment capacity of column. It can be obtained from moment curvature plot based on geometric properties of column. Knowing $V_{col-hinge}$,with some factor of safety γ1 requirement for V beam-hinge can be obtained. (γ1 may be assumed to be 0.8)

$$V_{capacity\ Beam\ hinge} = \gamma 1\ .V_{capacit\ column\ hinge} \tag{6}$$

6.2 Limit State of Connection Shear Capacity

It is essential that joint does not fail prior to beam or column. Hence equivalent story shear capacity of joint, must be greater than $V_{capacit\ column\ hinge}$. Principal stresses are reliable criteria for accessing performance of the joint (Pampanin, Calvi, & Moratti, Seismic Behaviour of R.C.Beam Column joints Designed for gravity loads, 2002). Critical value of principal stress (f_{pc}) is given as $f_{pc} = 0.29\sqrt{f_{ck}}$ for exterior connections [30]. Equivalent story shear for connection hinging in terms of β is defined as,

$$V_{capacit\ joint} = \frac{A_e\sqrt{\left(f_{pc}^2\right) - \frac{f_{pc}P}{A_c}}}{1 - \frac{\beta H_c}{(L_{Bc}+d_c)tan\alpha} - \frac{H_c(L_{Bc}-2X)}{j(L_{Bc}+d_c)}\left(1 - \frac{\beta d_b}{2Ltan\alpha} + \frac{(1-\beta)X}{L}\right)} \tag{7}$$

Where, $L = L_{Bc} - 2X$ and 'P' is axial load on column

Knowing $V_{capacity\ column\ hinge}$ from Eq. (5) and assuming factor of safety γ2 , required $V_{capacit\ joint}$ can be obtained for a value of β from Eq. (7).

6.3 Ensuring Beam Hinging Prior to Any Other Failure

In order to ensure beam hinging moment at haunch, max. moment in beam (Mbmax) must reach yield moment capacity Mpb of beam. Thus, equivalent story shear corresponding to beam hinging is given as,

$$V_{capacit\ beam\ hinge} = \frac{M_{pb}\left(1 + \frac{d_c+2X}{L_{Bc}-2X}\right)}{H_C} \tag{8}$$

Value of 'X' needs to be so adjusted that Eq. (6) is justified.

6.4 Thickness of Haunch Plate

Approximate thickness of haunch diaphragm required for K_{ten} is obtained as follows (Fig.7)

$$t = K_{ten}.\frac{L_{he}}{w_h E_s} \tag{9}$$

L_{he} and w_h are width and length of shaded portion of haunch plate in Fig. 7)

(E_s is Modulus of elasticity of haunch material)

Total thickness required t is divided in two diaphragm plates. These two diaphragm plates are separated by distance 'Z_h' and are connected through lateral bolts in order to fabricate an integrated 3-dimensional haunch. Details of bolting are illustrated in Fig. 7.

7. Effect of Haunch Retrofit Solution on As Built Connection

Numerical analysis of as built and haunch retrofitted subassembly is done to ascertain efficacy of proposed solution. A similar type of performance assessment is done by Pampanin et al. [41]. A two-dimensional beam-column subassembly is modelled in Finite Element analysis (FEA) software Etabs. The sectional properties of beam and column are as depicted in Fig.9. The column is 3.2 m in height and beam is 2 m in span. Load is applied at the tip of the beam. Displacement controlled loading protocol along with plastic hinge assignment on the subassembly is shown in Fig. 4. Haunch was modelled as spring with stiffness 150000 kN/m as illustrated in Fig. 4. Performance evaluation of as built and retrofit subassembly is shown graphically in Fig.5. The pattern of graph resembles to that presented in Pampanin et al. [41]. Lateral load carrying capacity of retrofitted subassembly increases by 30 % as compared to that of the as built specimen. The energy dissipation is calculated by estimating area under hysteresis loops. It is observed that haunch retrofit solution increases resilience of the subassembly by 27% and initial stiffness by 30 %. It is also noted that haunch retrofit solution ensures formation of plastic hinge with-in portion of beam beyond the outer-face of diagonal haunch. In addition, beam side sway mechanism is ensured in retrofit assembly as compared to joint shear side sway mechanism causing soft storey effect in case of as built subassembly.

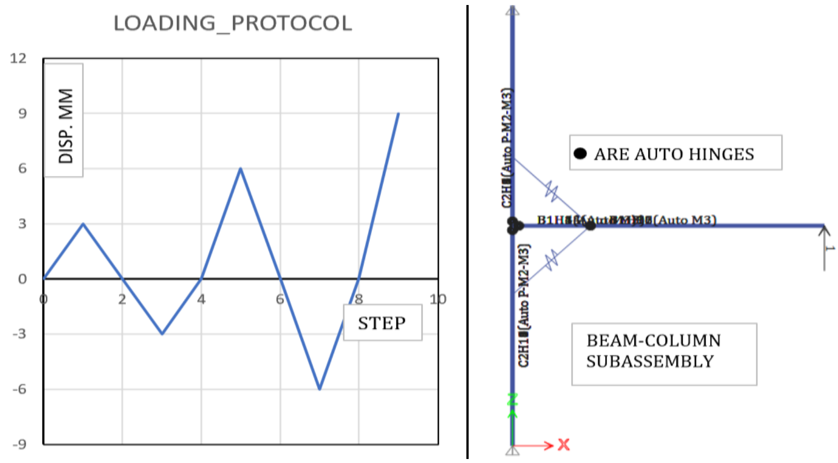

Fig. 4 Loading protocol and 2 -D subassembly model with plastic hinge assignment

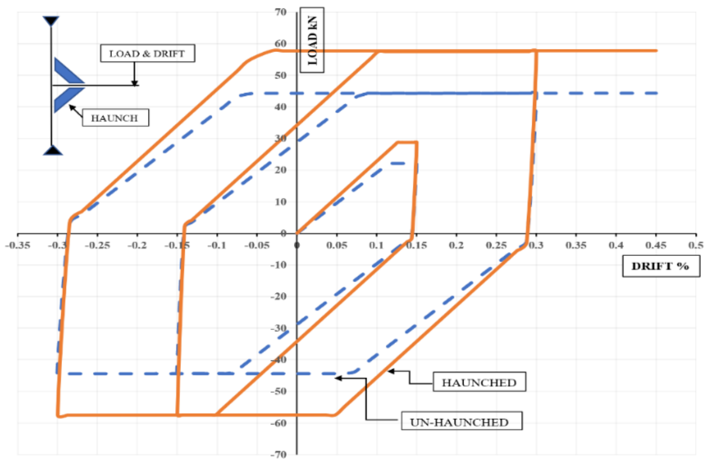

Fig. 5. Effect of retrofitting on performance of connection subassembly (Drift calculated as (displacement/2000)*100%)

8. Optimum Value of Haunch Stiffness

In case of ill detailed connections, it is essential to ensure that inadequate anchorage of beam reinforcement does not cause flexure failure at joint due to slipping of reinforcement, before formation of plastic-hinge, at the location of haunch in the beam. It is generally observed that beam bars are anchored in connection between 60 to 70 % of required development length. Thus, maximum beam moment capacity at the column-face is calculated to be 0.6 M_{pb}. (M_{pb} is plastic moment capacity of beam for fully developed reinforcement, which can be obtained from moment curvature relation for given properties of beam). Eq. (1) can be rewritten for

$$M_{bmax} = M_{pb} \ as,$$

$$M_{bc} = 0.6\,\gamma 1\, M_{pb} = M_{pb}\left[1 - \frac{\beta d_b}{(2\,L\,Tan\alpha)} + \frac{(1-\beta)X}{L}\right]$$

$$for\ \gamma 1 = 0.8\ and\ \alpha = 45 \quad \beta = \frac{L+2X}{d_b+2X} \tag{10}$$

Value of β from Eq. (10) is substituted in Eq. (4) to calculate optimum stiffness of haunch for given set of geometrical and material properties of the structure. It is observed that further increase in stiffness of haunch, beyond this value, does not have considerable effect on performance of the retrofitted assembly.

Fig. 6 illustrates a set of lateral loads against displacement curves obtained by numerical analysis for different values of haunch stiffness for same geometrical and material properties of a structural subassembly. It is observed that any increase in stiffness beyond 150000 kN/m, does not have substantial effect on performance of retrofitted subassembly for given set of structural parameters.

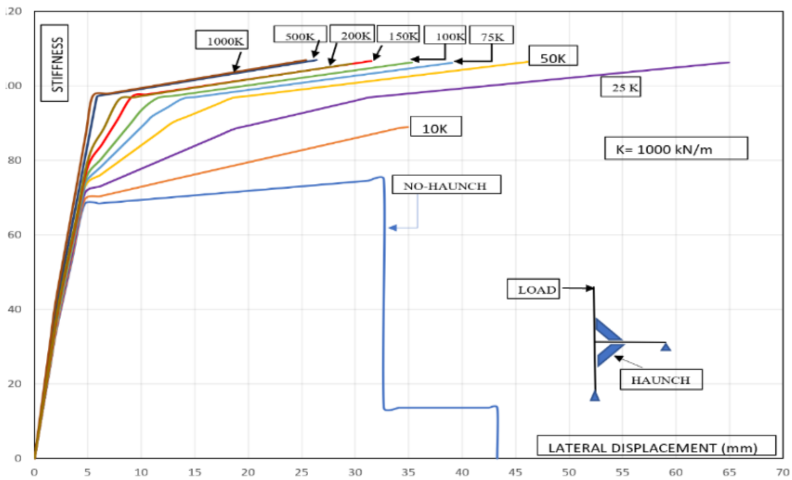

Fig. 6 Performance of joint subassembly for different haunch stiffnesses

9. Geometric Design of Haunch

Buildings with parking floor are most vulnerable during earthquakes owing to soft story effect. While designing the haunch it is essential to ensure adequate headroom below haunch element. Vertical projected length 'Y' (Fig.3) of haunch should be such that minimum headroom of 2.1 m is maintained below center of the haunch.

Higher values of α produce more efficient haunches [30]. how-ever, 'α' and headroom are inversely proportional depth of 600 mm. For such cases, practicable geometric dimensions of haunch are as shown in Fig. 7. Numerical investigations, done during this work, indicate that, α=45o yields a reasonably efficient haunch. Another parameter affecting performance of haunch is its axial stiffness, (Kd). Kd can be determined experimentally or numerically.

If Kd is different in tension and compression, then average of the two is taken for calculation of β. For most of the buildings parking floor height is 3.15 m with average beam depth of 600 mm. For such cases, practicable geometric dimensions of haunch are as shown in Fig. 7.

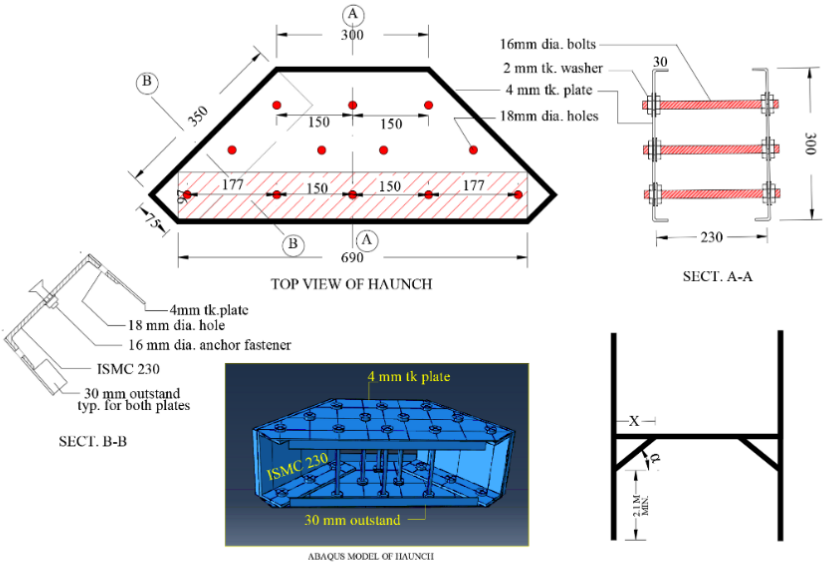

Fig. 7 Details of 4 BDH Haunch (4 BDH to indicate 4 mm thick haunch plates)

10. Comparison Between Single Plate and Equivalent BDH Solution

Comparative performance analysis of a 8 mm thick Single plate haunch (Genesio, 2012) as shown in Fig. 8 and a BDH made of two, 4mm thick diaphragm plates as shown in Fig. 7is done in FEM analysis software ABAQUS. Axial, Lateral, and torsional stiffnesses of both haunch elements are evaluated. Results of this analysis are presented in Table 1. constitutive properties of haunch materials are illustrated in Table 3.

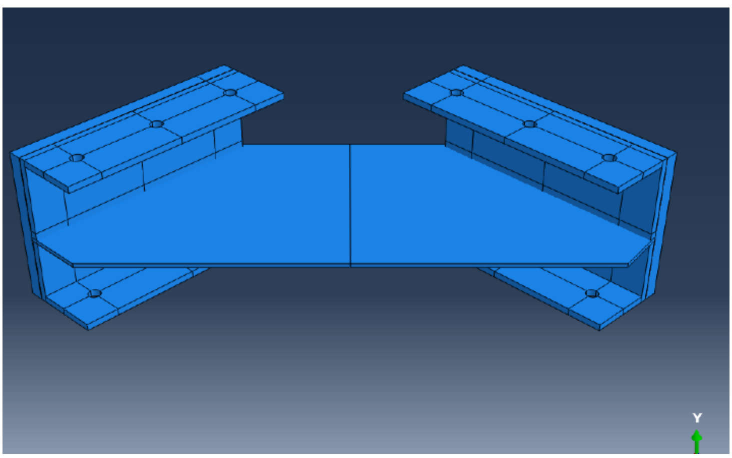

Fig. 8 Single Plate 8 mm tk. Welded Haunch

Table 1. Comparison between single plate and BDH Haunch

Parameter	Single plate 8 mm thick	BDH 4 mm tk. plates	% Rise
Comp. yield strength	313 kN	497 kN	59
Ulti. Comp. strength	446 kN	580 kN	30
Comp. stiffness initial	743000	662000	-12
Secant stiffness @ 3mm axial displacement	46505 kN/m	104500 kN/m	124
Tensile yield strength	389 kN	523 kN	34
Tensile ultimate strength	504 kN	636 kN	26
Torsional Stiffness	40kNm/rad	151kNm/rad	277
Torsional yield strength	3 kN m	12 kN m	300
Avg. energy dissipation @ 3 mm displacement	653050 N-mm	1267164 N-mm	94

11. Case Study (Parking+6 Storey Frame)

11.1 Details of As Built Structure

The efficacy of proposed retrofit strategy is assessed by push over analysis of a parking + 6 story 2-D frame, designed and detailed only for gravity loading as per IS 456-2002 (BIS, 2002). Geometrical and structural details of frame are illustrated in Fig. 9. Infill walls with less than 20% opening have been modelled as diagonal struts for habitable floors as shown in Fig. 9.

Various parameters and constitutive properties required for design of a haunch are presented in Table 2. It is assumed that beam longitudinal rebars are anchored in column core for 65% of required development length. Thus, effective area of a 16 mm diameter bar available as a fully developed bar is 201x 0.65=131 mm^2.So, support section of beam is provided with 131 mm^2 for each reinforcement bar.

All beam and column ends are modelled with default non-linear hinge parameters (ASCE 41-17) as available in FEM analysis software Etabs.

Table 2. Parameters and constitutive properties of as built structure for seismic assessment

Concrete module Elastic/Shear	22360 MPa /9316 MPa
Distance between top and bottom bars of beam (j_d)	390mm
c/c height of column (H_c)	3150 mm
Effective joint area ((A_j)	121900 mm^2
c/c beam span (L_B)	5000 mm
Clear beam span (L_C)	2235 mm
Diameter of main reinforcement	16 mm
Effective depth of beam(db)/column(dc)	420/500 mm
Area of shear reinforcement leg (Asv)	100 mm^2
Limit state of principal stresses $Pt=0.29\sqrt{f_{ck}}$	1.296 MPa
Spacing of shear reinforcement for beam/column	120/150 mm
Yield strength of main reinforcement (f_y)	415 MPa
Yield strength of shear reinforcement (f_s)	415 MPa
Maximum moment capacity of beam (M_{bmax})	73 kN-m
Maximum moment capacity of column (M_{cmax})	169 kN-m

Displacement controlled Push Over analysis is carried out to access performance of the structure while being subjected to lateral loading.

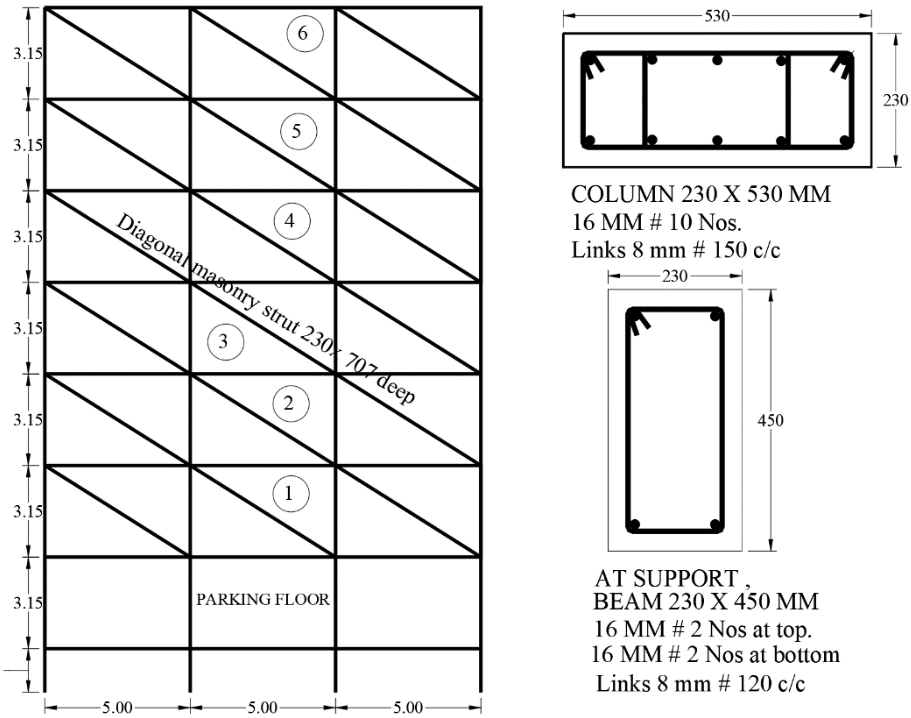

Fig. 9 Details of as built structure (Same sections are used for assessment of subassembly in Fig.4)

11.2 Modelling of Brick Masonry

Infill masonry walls are modelled as diagonal struts with hinged ends connected at beam-column connections as shown in Fig. 9. Pore [43] has done extensive study on constitutive properties of brick masonry across India. Strength of masonry(σ_p) for 1:6 mortar given by [43]is considered in this study.

$$\sigma_p = 0.175[\sigma_{cb}^{1.22} + \sigma_{mo}^{0.2}]$$

It is also observed (Pore, 2007)that compressive strength of brick (σ_{cb}) across India varies to a great extent. Except for Uttar Pradesh and West Bengal, strength of brick varies between 3 to 10 N/mm^2. For western Maharashtra region assuming, $\sigma_{cb} = 5\ N/mm^2$ and $\sigma_{mo}(strength\ of\ mortar) = 6\,N/mm^2$ (Pore, 2007) for 1:6 mortar

(For 1:6 mortar) Strength of masonry is calculated as, $\sigma_p = 1.5\frac{N}{mm^2}$.

IS 1893:2016 (BIS-IS-1893, 2016) gives value of elastic modulus of brick masonry as $E_m = 550\ \sigma_p$ and width of strut as $w_{ds} = 0.175\ \alpha_h L_{ds}$

Clause 7.9.2.2 of IS 1893:2016 (BIS-IS-1893, 2016) may be referred for further details of the terms mentioned above. Modulus of elasticity and width of strut for assumed data of prototype structure are calculated as 825 N/mm^2 and 707 mm respectively.

11.3 Performance of As Built Structure

Push over analysis of as built structure reveals formation of connection hinges (Fig.10) at parking storey slab level leading to soft storey mechanism. All upper habitable floors do not develop any type of non-linear hinge owing to diagonal masonry struts. Detailed results of push over analysis are discussed in later sections.

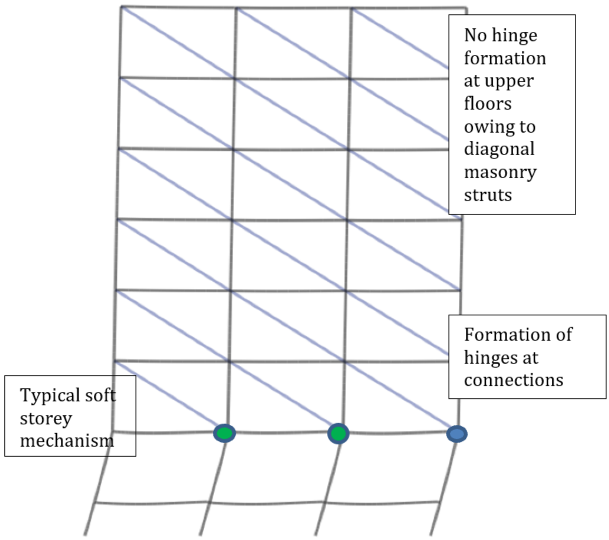

Fig. 10 Hinge formation at parking floor in as built frame

11.4 Retrofit Strategy

Based on performance of as built structure, it is decided to retrofit parking slab beam column connections by BDH elements. Eq. (9) and Table 2 yield β= 3 For required strength hierarchy. Substituting β= 3 in Eq. (4), optimum haunch stiffness is calculated as $K_{d\ av}$= 380000 kN/m. Referring to Eq. (10) and details of haunch shown in Fig. 7, thickness of haunch diaphragm works out to be 4 mm on each side. 16 mm(M16) diameter nominal lateral confining bolts are used to assemble the BDH unit.

11.5 Stiffness of Haunch Assembly

Analytical calculation of compressive and tensile stiffness is difficult due to non-standard geometry and overall configuration of the 3-D haunch. It is observed in literature [31] that it is a common practice to assume stiffness of haunch element only. Effect of joineries is usually accounted for by assuming a certain factor of safety. In this work numerical analysis software ABAQUS is used to determine the actual stiffnesses of haunch assembly. Numerical model of haunch during compression test is shown in Fig.11. All the elements are modelled as 3-D solids. Constitutive properties of haunch material are presented in Table 3. All components are meshed with C3D8R Brick elements with hour-glass control. Mesh Size of each element is adjusted, to properly maintain hierarchy between slave and master components. Diaphragm plate was discretized with 15 mm size mesh element and so on. Boundary conditions were imposed to avoid over-constraining as well as numerical singularities during analysis. Displacement controlled Ramp load with a limiting value of 6 mm was applied to cause axial deformation of BDH element. Static general analysis procedure was adopted. The Compressive and tensile load carrying capacities of haunch are presented in Fig. 11. Preliminary analysis of frame with nominal stiffness of 400000 kN/m indicates axial displacement of haunch element as 1.25 mm. Thus, modelled haunch should retain a stiffness of at-least 400000 kN/m at 1.25 mm displacement. Table 3 gives values of load carrying capacity of haunch assembly against displacement for tension and compression. It is observed that lowest stiffness of proposed haunch geometry is 480000 kN/m in compression (Table 4).

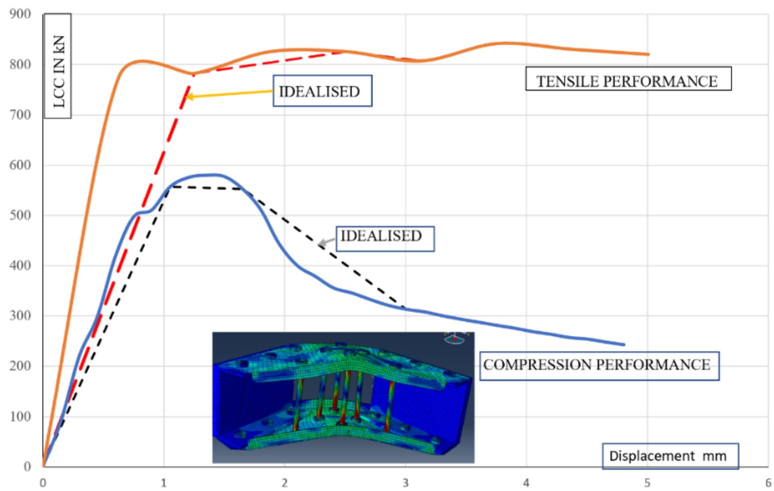

Fig. 11 Axial load carrying capacity of BDH element. (LCC on 'Y' axis indicates axial Load Carrying Capacity of the BDH element)

Table 3. Constitutive material properties for BDH specimen in Fig.7

Properties of diaphragm plate		Properties of M16 bolts	
E= 2e5 MPa	fu=450 MPa	E= 2e5 MPa	fu=827 MPa
fy=250 MPa	Poisson's ratio=.3	fy=640 MPa	Poisson's ratio=.3
stress	plastic strain	stress	plastic strain
250.8125	0	643.328	0
271.2852405	0.001504126	662.9481015	0.001241986
292.0436072	0.003777478	683.0245297	0.003082307
313.2355569	0.007138458	703.7899065	0.005800029
335.0687107	0.011998233	725.5847446	0.009788564
357.8281624	0.018874993	748.9024348	0.01559459
381.8979864	0.028405786	774.4510102	0.023965632
407.7869165	0.041353305	803.2372476	0.035904932
436.1587006	0.058604279	836.6802921	0.052729014
467.8676583	0.081155536	876.7640328	0.076118091
504	0.110083955	926.24	0.108142159

11.6 Modelling of Haunch in Etabs

A haunch assembly is made of two different types of link elements in Etabs (Fig.13)

- Rigid link for connecting basic haunch element to beam column centerlines
- Elasto-plastic haunch modelled as non-linear link element having constitutive properties derived from FEM analysis and illustrated in Table 4

Table 4. Stiffness variation of haunch against axial displacement

Displacement (-) is compression mm	Load in kN	Stiffness available kN/m
-3	-300	100000
-1.65	-570	345000
-1.2	-577	480000
0	0	0
1.2	780	650000
2.5	825	330000
3	750	250000

11.7 Analysis of Frame

Modal push over analysis for first mode of vibration and a response spectrum analysis as per IS 1893:2016, in global X direction, is done to access performance of as built and retrofitted frames.

12. Results and Discussion

(For numerical values of parameters in the formulae please refer Table. 2)

To understand need for retrofitting, it is essential to establish strength hierarchy of as built frame. It is done by comparing inter-storey column shear requirements for formation of plastic hinges in column and beam.

Column storey shear required to form a plastic hinge in beam (V_CHB) is given as,

$$V_{CHB} = 2\frac{M_{BMax}}{L_{BC}} = \frac{73}{2.235} = 32.66\ kN$$

Column storey shear required to form a plastic hinge in column (V_CHC) is given as,

$$V_{CHC} = \frac{L_B V_{CHB}}{2H_C} = \frac{5 * 32.66}{2 * 3.15} = 25.92\ kN$$

$$actored = (0.7 * 25.92) = 18.14\ kN$$

Column storey shear required to form plastic shear hinge in joint (V_CHJ) is given as

$$V_{CHJ} = \left(\frac{M_{BMax}}{jd}\right) - V_{CHC} = 132.36kN$$

$$factored(0.7 * 0.7) = 64.68\ kN$$

Thus, $V_{CHC} < V_{CHB} < V_{CHJ}$. Joint shear capacity is more than other shear capacities however this hierarchy indicates column hinge formation at a connection, before beam hinge formation and hence is not acceptable. More-over, numerical analysis reveals formation of flexural hinges at connections (Fig. 10). So, connections at parking slab level need to be strengthened by BDH element.

12.2 Analytical Assessment of Retrofitted Frame

It is decided to adopt a BDH element with β=3 as already discussed. Inter-storey shear capacities are calculated as follows

Shear capacity for column hinging

$$V_{CR_{COLUMN_{HINGE}}} = \frac{M_{CMax}}{\frac{H_C}{2} - \frac{d_b}{2} - X\,Tan\alpha} = 192.82\ kN$$

Factored capacity = 0.7*192.82 = 134.97 kN

Shear capacity for beam hinging

$$V_{CR_{BRAM_{HINGE}}} = \frac{M_{BMax}}{H_C} * \left(1 + \frac{d_c+2X}{L_{BC}-2X}\right) = 105\ \text{kN}$$

Shear capacity of Joint hinging is given as,

$$V_{CR\ JOINT\ HINGE} = \frac{A_c\sqrt{(f_{pc}^2) - \frac{f_{pc}P}{A_c}}}{1 - \frac{\beta H_c}{(L_{Bc}+d_c)tan\alpha} - \frac{H_c(L_{Bc}-2X)}{j(L_{Bc}+d_c)}\left(1 - \frac{\beta d_b}{2Ltan\alpha} + \frac{(1-\beta)X}{L}\right)}$$

= -91 kN factored (0.7*-91) = -63.7 kN.

Negative sign indicates shear force will be required from opposite direction to cause tensile failure of joint. β>1 will always yield negative shear capacity at joint (Fig.3). Having negative capacity is even better for protection of connection [30].

Thus,

$$V_{beam-hinge-capacity}(as\ (-)ve) < factored\ V_{column-hinge-capacity} < factored\ V_{joint-capacity}$$

indicating proper strength hierarchy being maintained after retrofitting of the prototype structure.

12.3 Comparative Push Over Analysis

Comparative results of displacement-based push over analysis for first mode are plotted in Fig. 12.

Table 5. illustrates comparison of un-haunched and haunched frames based on key performance parameters. Location of first 'Immediate Occupancy'(IO) hinge formation is plotted on load-displacement graph. It is observed that, there is a substantial increase in the performance parameters of retrofitted frame over those of as built frame.

Table 5. Comparison of push over performance of as built and retrofitted frames

Parameter	Un-Haunched Frame	Haunched Frame	% Change
Ultimate lateral force capacity	417 kN	594 kN	+ 42
Lateral load at yield point	247 kN	464 kN	+ 87
Energy dissipation till formation of first LS hinge	38.55 kN-m	52.34 kN-m	+36

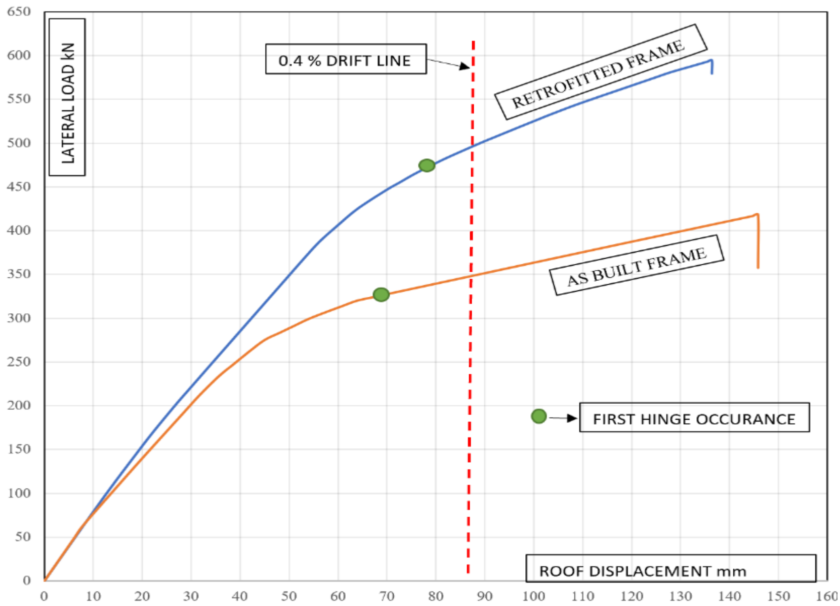

Fig. 12 Comparative push over analysis

Fig.13 shows hinge formation in retrofitted frame. It is observed that all hinges form only at haunch location and none of the hinges are formed in the connection region.

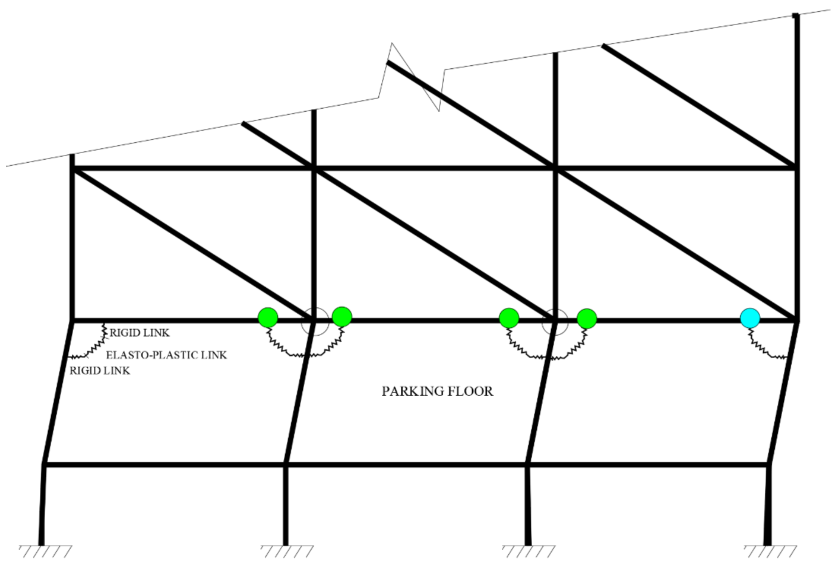

Fig. 13 Hinge formation at parking level in retrofitted structure (Green indicate IO hinges and cyan indicate LS hinges)

12.4 Comparative Seismic Analysis

Haunched and un-haunched frames are subjected to response spectrum loads as per IS1893:2016 [44]. Both linear and non-linear response spectrum analysis was carried out.

Table 6. illustrates comparative performance of as built and retrofitted structures while subjected to same seismic loads. It is observed that performance of retrofitted frame is better than as built frame with reference to maximum roof displacement and maximum storey drift by 14.6 and 42 % respectively. In case of non-linear response spectrum analysis, it is observed that as built frame yields much before retrofitted frame and maximum roof displacement and drifts for as built frame are more by 106 % and 154 % respectively. It is also observed that for as built structure the hinges form at connections whereas in case of retrofitted structure, hinges were formed only at the haunch-beam interface. Graphical performance of as built and retrofitted frames for non-linear response spectrum analysis is shown in Fig. 14.

Table 6. Comparative results of response spectrum analysis

Response spectrum analysis (Linear performance)			
Parameter	As built frame	Retrofitted frame	% Change
Max. roof displacement	51 mm	45 mm	- 14.6
Max. storey drift	4.7e-3	3.3 e -3	- 42
Response spectrum analysis (Nonlinear performance)			
Max. roof displacement	126 mm	61 mm	-106
Max. storey drift	2.7 e -3	1.06 e -3	- 154

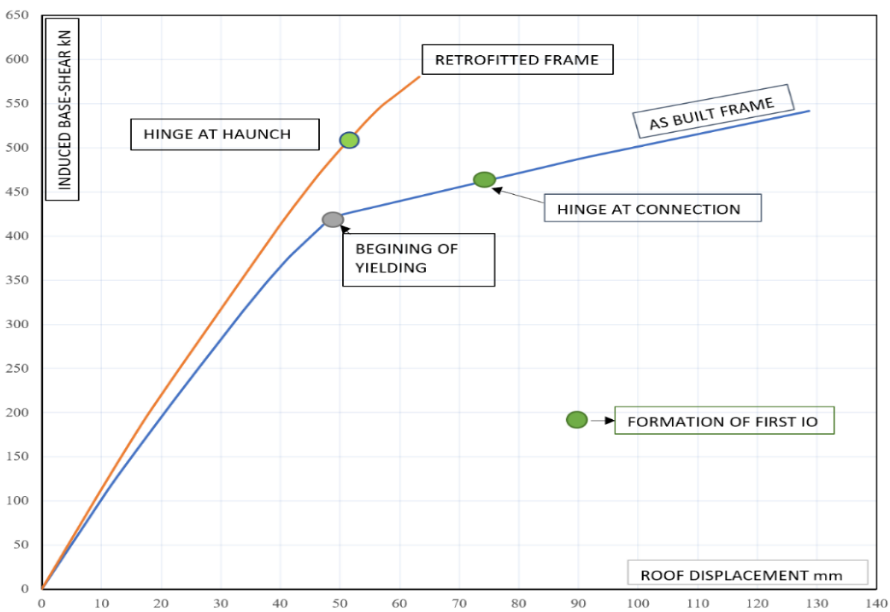

Fig. 14. Non-linear performance for as built and retrofitted structure

13. Experimental Validation of Retrofit Solution

A haunch retrofit solution was designed based on the methodology proposed in in this paper. Experiments on 1/3 scaled down specimen were carried out [39]. Details of haunch designed for the experimental program are shown in Fig.14. Abstract of experimental finding is depicted in Fig. 15. For details of experimental verification procedures [39], may be referred.

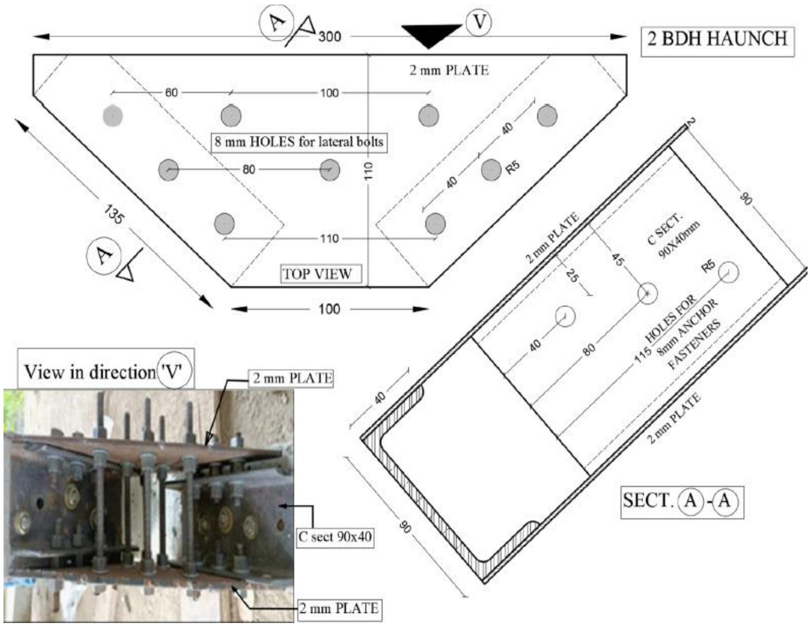

Fig. 14 Details of haunch used for experimental verification [39]

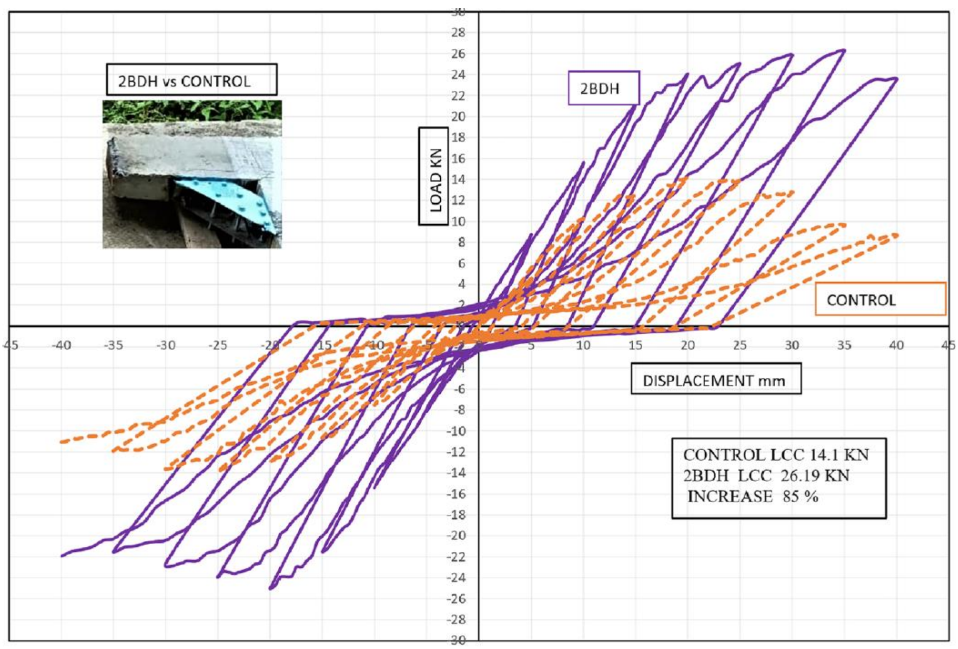

Fig. 15 Performance enhancement due to retrofit haunch [39]

Table 7 presents abstract of performance enhancement due to use of proposed BDH retrofit solution. [39]

Table 7. Performance evaluation of retrofit solution (Experimental)

Parameter →	Initial stiffness kN/mm	Resilience kN-mm	ductility	Load carrying capacity kN
Control (As built specimen)	0.962	20.39	1.13	14.10
2-BDH Retrofitted specimen	1.67	49.5	2.67	26.19
% Rise	85	94	136	85

14. Conclusions

Ill-detailed frames undergo critical brittle failure in joint region due to lack of capacity design considerations.

Modelling brick masonry as equivalent diagonal struts leads to formation of soft storey mechanism at parking floor. Hinges at connections are developed at parking storey slab level inducing a pan cake failure. Introduction of haunch re-routs critical stress path in subassembly excluding joint panel location.

Numerical analysis confirms that introduction BDH element near beam column connection substantially improves overall performance of beam-column connection subassembly. BDH retrofit unit has 277 % more torsional stiffness, 94 % more resilience and 59 % more yield strength, as compared to an equivalent single plate haunch. Higher yield strength ensures that BDH remains in elastic zone for higher loads, imparting higher resilience to beam-column connection subassembly.

A BDH element for given set of structural data can be designed and detailed adopting step by step flow-chart illustrated in this paper.

Elasto-plastic, bolted nature of BDH element enhances overall resilience of the structure.

A study of performance enhancement of retrofitted subassembly for different haunch stiffnesses reveals that, there is an optimum level of haunch stiffness for given set of structural properties, beyond which any increase in haunch stiffness has very marginal effect on performance of retrofitted beam column subassembly.

A BDH element not only shifts formation of plastic hinges in beam, away from connection region but also improves load carrying capacity, resilience, and rotational stiffness of connection subassembly.

The push over and seismic performance analysis of a parking+6 storey frame, designed for gravity loads, with and without retrofitting underlines advantages of retrofit strategy. It is observed that lateral load capacity at yield point of retrofitted frame increases by 87%, Ultimate lateral load capacity by 42% and resilience by 36 %. Comparative non-linear response spectrum analysis of retrofitted frame reveals that maximum roof displacement is reduced by 106 % and maximum storey drift by 154 %. Experimental investigations on 1/3 scaled down specimens indicate increase of 85 % in load carrying capacity,136 % increase in ductility and 94 % rise in resilience.

It is concluded that this performance enhancement is due to (i) protection of exterior beam column joints (ii) rerouting of critical stress path excluding connection region (iii) reduction of inter-storey drift and maximum roof displacement for given seismic excitation.

Scope for Further Study

BDH elements connected to parent structure through anchor fasteners tend to exhibit slip in anchors subjected to reversed cyclic loading. A better anchoring technique needs to be investigated.

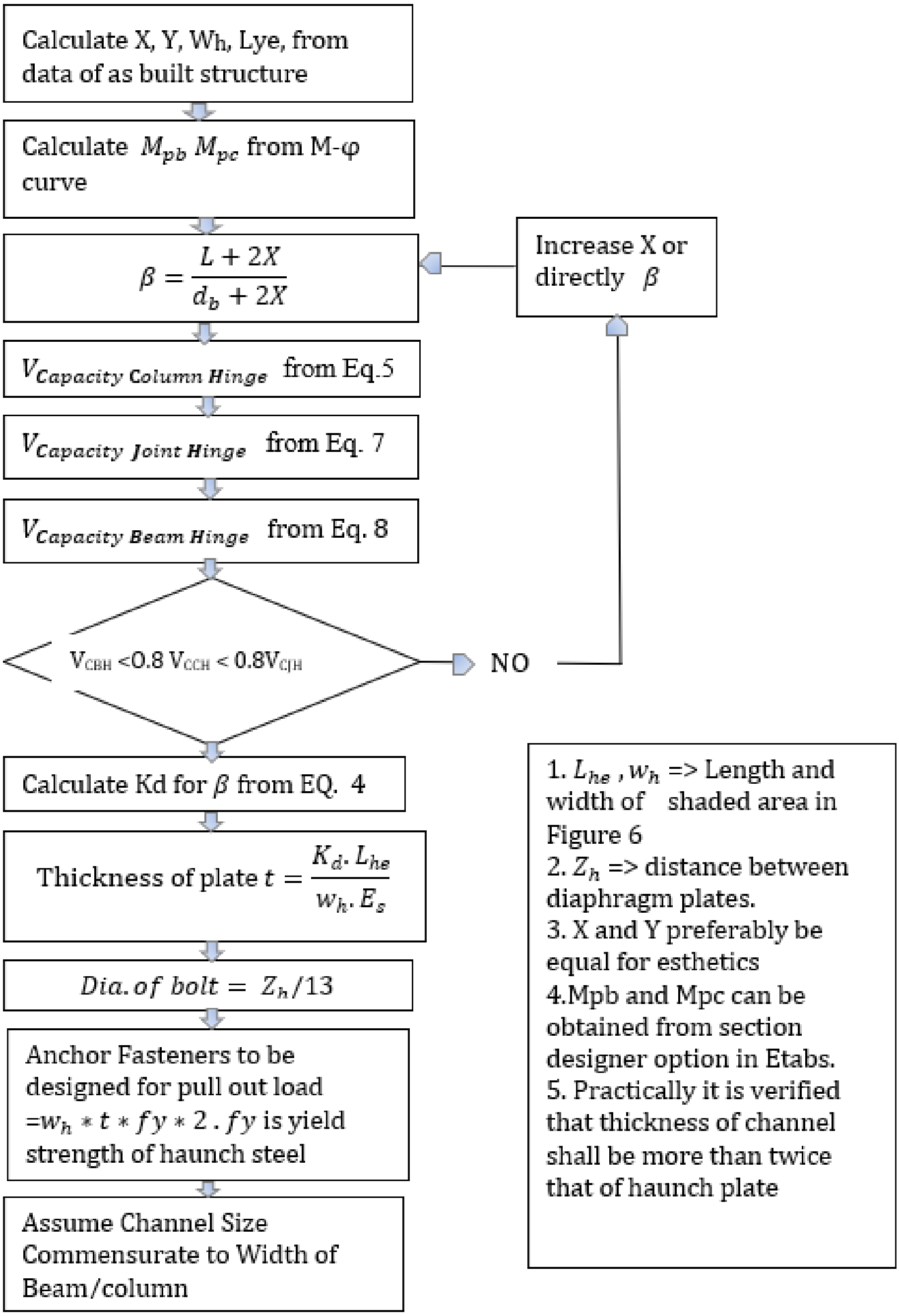

Fig.16 Flow Chart for Design of BDH Element

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