Assessment of code-based design approach of buckling restrained braced frames in P100-1/2013 using pushover analysis

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Abstract. Buckling restrained braced frames (BRBFs) are a relatively new structural system, characterized by high seismic energy dissipation. The Romanian seismic design code P100-1/2013 has recently introduced design provisions for buckling restrained braced frames. According to the code, the dissipative components are designed for the seismic forces reduced by behavior factor q, while the non-dissipative components are capacity-designed for seismic forces amplified with an over-strength factor $\Omega'$. The objective is to assess the code-based capacity design procedure using the nonlinear static analysis. Three structural configurations are considered: two BRBF configurations and a dual BRBF configuration. The structures are first designed following the code procedure, and then analyzed using a pushover analysis, using a numerical model of the buckling restrained braces calibrated on experimental data.

1. Introduction

It is known that conventional brace members utilized to reduce the lateral displacements on structural buildings, when subjected to compressive forces exhibit buckling deformation and their hysteretic behavior is unsymmetrical in tension and compression. To solve this, engineers developed a new type of braces, the buckling-restrained brace (BRB), which due to its construction exhibits the same behavior in both tension and compression.

The buckling-restrained braced frame (BRBF) system differs from a conventionally brace frame in that the BRB braces are capable to yield in both tension and compression and are more ductile.

Sabelli, Mahin and Chang [1] showed that the seismic performance of BRBFs have comparable or better behavior than conventionally braces.

Uang and Kiggins [2] studied whether the use of a backup moment frame system, which provides a restoring force mechanism, in a dual system can minimize residual deformations.

Studies about design of BRBFs implementation in Romania were made by Stratan et all [3].

The objective of this paper is to perform an assessment for three structure configurations: a dual antiseismic structural system consisting from moment resisting frames (MRF) combined with buckling restrained braced frames (BRB) and two buckling restrained braced frames (BRBF) with base of central columns fixed or pinned using pushover analysis. To achieve this we developed a finite model of the structure and calibrated the numeric model of BRB using an experimental model of BRB from IMSER project [6] data.
2. Prototype structure

2.1. Location
The location for this structure is city of Timisoara, Romania. The ground motion for this place has an acceleration of 0.20 g. The seismic action to be taken into account for the “Life Safety requirement” corresponds for a return period of 225 years and has a probability of exceedance of 20 % in 50 years.

2.2. Structural solution
As it can be seen in Figure the structures has the following particularities: Plan dimensions - 37.50 x 22.50 m, Spans on transversal direction - 3 x 7.50 m, Spans on longitudinal direction - 5 x 7.50 m, Story height - 3 x 3.50 m, Category - Office areas, Type of steel elements utilized in the structure - European hot-rolled steel sections, Steel grade for elements - S355. For exterior walls is utilized lightweight glass curtain wall system. The partition walls in interior are fabricated from gypsum boards fastened with screws on a self-supporting lightweight metal structure with infilling thermal insulation.

As Figure presents, we analyzed three structures. The lateral load resisting system consists from one inverted V buckling restrained braced frame (BRBF) for each of them. The BRBF is used in “X” direction in axes “1” and “6”, and in “Y” direction in axes “A” and “D”. First analyzed structure is a dual moment resisting frame (MRF) combined with BRBF - (D-BRBF) system.

For second and third structure, the MRFs are replaced with pinned beams, the lateral resisting system is only the BRBF. The second structure consists from BRBF with central columns fixed - (BRBF-F). The third structure consists from BRBF with central columns pinned - (BRBF-P).

The other frames are designed only for gravity loading. The secondary beams are considered pinned to the main beams. For gravity loading was considered a 0.5 m console on the perimeter contour of the structure, plan dimensions for gravity loading being - 38.50 x 23.50 m.

Due to the fact the structure is regular, the structural elastic analysis has been performed on a 2D model for half of the structure and only the frame from axis 1 was analyzed.

The gravity load resisting system is formed from a reinforced concrete slab cast on a profiled steel sheeting. The floor is supported on main and secondary beams.

The last floor is designed as an open space terrace. The structure has a compact shape in the plane, the floor made from reinforced concrete has sufficient stiffness in its plane so can provide the diaphragm effect at each level. As a consequence, the structure is regulated in plane in accordance with the requirements of paragraph 4.4.3.2 of Romanian seismic code P100-1 [5].

The analyzed structure is monotonous on vertically, with no discontinuities, with insignificant variation of rigidity and vertical resistance.

The mases applied on the construction are distributed relatively uniform, falling within the 50% limit of variation between adjacent levels. Consequently, the structure is vertically regulated as required by paragraph 4.4.3.3 of P100-1 [5].

The sections of structural elements are presented in table 1.

| Structure type | Level | Braced frame beams | Unbraced frame beams | BRB sections [mm] | Central columns | Marginal columns |
|----------------|-------|---------------------|----------------------|------------------|----------------|-----------------|
| D-BRBF         | 3     | HE240A IPE360       | 15x50                | HE260B           | HE220B         |                 |
|                | 2     | HE240A IPE360       | 20x65                | HE260B           | HE220B         |                 |
|                | 1     | HE240A IPE360       | 20x70                | HE260B           | HE220B         |                 |
| BRBF-F         | 3     | HE240A IPE400       | 15x60                | HE220B           | HE200B         |                 |
|                | 2     | HE240A IPE400       | 20x65                | HE220B           | HE200B         |                 |
|                | 1     | HE240A IPE400       | 20x75                | HE220B           | HE200B         |                 |
| BRBF-P         | 3     | HE240A IPE400       | 15x60                | HE240B           | HE200B         |                 |
|                | 2     | HE240A IPE400       | 20x65                | HE240B           | HE200B         |                 |
|                | 1     | HE240A IPE400       | 20x75                | HE240B           | HE200B         |                 |
2.3. Design of the structure
The elastic design is performed for ultimate and serviceability limit states. The norm according to which the design combinations are considered is CR0 [4].

The check of structure elements begins with persistent design situation. After this the structure elements are checked in seismic design situation.

In seismic design situation, first are checked dissipative members (buckling restrained braces (BRB), and unbraced frame beams for dual MRF+BRBF - (D-BRBF) system), (buckling restrained braces (BRB) for BRBF-F and BRBF-P systems). In second order are checked non-dissipative members (braced frame beams and columns). The non-dissipative members are designed having an over-strength with respect to the buckling restrained braces, and unbraced frame beams for D-BRBF system, and with respect to the buckling restrained braces for BRBF-F and BRBF-P systems.

For dual MRF+BRBF - (D-BRBF) system, according to paragraph 6.10.2 (2) from P100 [5], the unbraced frames, located in the bracing direction of the building has to be able to undertake at least 25% of the seismic action in the hypothesis in which bracing frames came out of work.

Accordingly to this, moment resisting frames (MRF) should be able to resist at least 25 % of the total seismic force using general formula:

$$F_{y}^{MRF} \geq 0.25 \cdot \left( F_{y}^{MRF} + F_{y}^{BRBF} \right)$$  (1)
Using the formulas below can be determined the duality of this particular structure:

\[ 2F_{y}^{MRF} + F_{y}^{BRBF} = F_{y}^{DUAL} \]  
(2)

\[ 2F_{y}^{MRF} \geq 0.25 \cdot F_{y}^{DUAL} \]  
(3)

where \( F_{y}^{MRF} \) is the resistance of MRF; \( F_{y}^{BRBF} \) - the resistance of BRBF; \( F_{y}^{DUAL} \) - is the total resistance of MRF+BRBF structure.

The duality checks for the MRF+BRBF (D-BRBF) structure is represented in table 2.

| Level | Beam | Brace (x) | \( F_{y}^{BRBF} \) (kN) | \( F_{y}^{DUAL} \) (kN) | \( 2F_{y}^{MRF} \) (kN) | \( 0.25F_{y}^{DUAL} \) (kN) | Duality (%) |
|-------|------|-----------|--------------------------|--------------------------|--------------------------|--------------------------|-------------|
| 3     | IPE360 | 15x50     | 389.28                   | 802.70                   | 413.42                   | 200.67                   | 51.50       |
| 2     | IPE360 | 20x65     | 674.76                   | 1088.18                  | 413.42                   | 272.04                   | 37.99       |
| 1     | IPE360 | 20x70     | 726.66                   | 1140.09                  | 413.42                   | 285.02                   | 36.26       |

3. The numerical modelling

3.1. Design of the structure of the BRB

The nonlinear behavior of the BRB is very important, and in order to have a high level of confidence in nonlinear computation it is necessary to calibrate the BRB according to an experimental model.

For numerical BRB behavior was utilized the experimental data for a BRB model from IMSER project [6]. The geometric data of experimental model are according to figure 2.

\[ K_{ech} = (\Sigma(Ki)^{-1} + \Sigma(Ke)^{-1} + \Sigma(Kt)^{-1} + \Sigma(Kp)^{-1})^{-1} \]  
(4)

where: \( Ki \) - stiffness of joint zone; \( Ke \) - stiffness of elastic zone; \( Kt \) - stiffness of transition zone; \( Kp \) - stiffness of plastic zone.
For non-elastic calculus of buckling restrained braces, to take into account that the cross-sectional area varies along the length of the BRB, the equivalent stiffness of the member with constant section must be corrected by multiplying with the $k$ factor:

$$k = K_{ech} \cdot L_n / (EA_p)$$

(5)

where $A_p$ – plastic area of BRB steel core; $L_n = 5129.57 \text{ mm}$ - joint to joint length of buckling restrained brace.

The modelling of equivalent stiffness of BRB by an equivalent bar of constant section can be achieved by modifying the Young’s modulus ($E=210000 \text{ N/mm}^2$) of the steel assigned to the section: $E_{ech} = k \cdot E = 315000 \text{ N/mm}^2$.

According to experimental data obtained from IMSER project [6], the value of effective stiffness of buckling restrained braces factor is: $k = 1.5$.

The force-deformation diagram of buckling restrained brace (BRB) is shown in figure 3.

![Figure 3. Force-deformation diagram of BRB.](image)

The yield capacity at tension and compression:

$$T_y = P_y = \gamma_{ov} \cdot f_y \cdot A_p$$

(6)

Corrected capacity at compression according to P100-1-2013, paragraph 6.11.2(2):

$$P_{max} = \beta \cdot \omega \cdot \gamma_{ov} \cdot f_y \cdot A_p$$

(7)

Corrected capacity at tension according to P100-1-2013, paragraph 6.11.2(3):

$$T_{max} = \omega \cdot \gamma_{ov} \cdot f_y \cdot A_p$$

(8)

The utilized values of $\beta$, $\omega$ factor according to IMSER project [6] are: $\omega = 1.45, \beta = 1.17$.

The yielding capacity according to IMSER project [6] tests: $\gamma_{ov} \cdot f_y = 397.8 \text{ N/mm}^2$.

The deformation capacity in compression ($\Delta_{bm}$) and deformation in tension ($\Delta_{by}$) are calculated:

$$\Delta_{bm} = \varepsilon_{c,max}(\%) \cdot L_n \cdot (100\%)^{-1}$$

(9)

for ($\Delta_{bm}^{-}$) and ($\Delta_{bm}^{+}$) was adopted $\varepsilon_{c,max} = 2.6\%$ of total BRB length - $L_n$.

The yield capacity deformation ($\Delta_{by}$) is calculated:

$$\Delta_{by} = T_y \cdot L_n \cdot (E_{ech} \cdot A_p)^{-1}$$

(10)

The BRB behavior input data for model calibration was according to table 3.
Table 3. BRB calibration input values.

| $A_p$ (mm²) | $P_{\text{max}}$ (kN) | $T_{\text{max}}$ (N) | $T_y$ (kN) | $\Delta_{by}$ (m) | $\Delta_{b/2}$ (m) | $\Delta_{bm}$ (m) |
|-------------|----------------------|---------------------|-----------|------------------|-------------------|----------|
| 840         | 566.889              | 484.520             | 334.152   | 0.00648          | 0.1334            | 0.1334   |

The numeric calibration of BRB behavior was made in a finite element model build in SAP2000 software which include a truss with length $L = L_n = 5129.57$ mm. Using the parameters above was assigned a plastic axial P hinge at half-length of the brace.

The characteristics of BRB hinge introduced in SAP2000 software for numeric calibration, according to Table.

The comparison between experimental and numerical curves of BRB is presented in figure 4.

![Figure 4. Force-deformation curves of experimental and numerical BRB model.](image)

The cyclic experimental behavior model is quite similar with monotonic numerical model. The numerical monotonic model is used for non-linear static analysis.

3.2. Numerical model of the structure

The nonlinear model of the structure is a 2D model. Second order effects acting on interior gravity frames were modelled using a leaning column. The leaning column is considered pinned at each level and is linked to the structure by a rigid diaphragm constraint.

Concentrated plasticity was used to model the nonlinear response of the structure. The assigned plastic hinges for beams are type M3, for columns are type P-M3, and are assigned at each end of frames. The assigned plastic hinges for BRB braces are type P, at half-length of the brace.

4. Non-linear static analysis

4.1. Non-linear static analysis procedure

The non-linear static analysis is a two-step analysis. In the first step, the gravitationnal loads are applied on the structure. The lateral loads are applied in the second step.

4.1.1. Attribution of modal distribution of forces.

The static non-linear analysis is performing by use of a modal distribution of forces.
In the modal distribution of forces, forces are computed in function of the first mode of vibration of the structure. The forces are calculated with formula:

\[ F_i = m_i \cdot \phi_i \]  

where: \( F_i \) is the force to the assigned on each storey; \( m_i \) is the mass of each storey; \( \phi_i \) is the deformation of each story.

The forces from modal and uniform distribution are shown in table 4.

| Structure type | Modal Level | \( m_i \) [t] | \( \phi_i \) [-] | \( F_i \) [t] | \( F_i \) [-] |
|----------------|-------------|---------------|----------------|-------------|-------------|
| D-BRBF        | 3           | 310.33        | 1.000          | 310.331     | 1.000       |
|                | 2           | 298.34        | 0.682          | 203.593     | 0.656       |
|                | 1           | 298.34        | 0.336          | 100.321     | 0.323       |
|                | 3           | 310.33        | 1.000          | 310.331     | 1.000       |
| BRBF-F        | 2           | 298.34        | 0.686          | 204.532     | 0.659       |
|                | 1           | 298.34        | 0.329          | 98.201      | 0.316       |
|                | 3           | 310.33        | 1.000          | 310.331     | 1.000       |
| BRBF-P        | 2           | 298.34        | 0.692          | 206.595     | 0.666       |
|                | 1           | 298.34        | 0.345          | 102.967     | 0.332       |

### 4.1.2. Attribution of plastic hinges

The behavior input data for BRB braces plastic hinges due to experimental model calibration was according to table 5.

| Structure type | Level | Hinge name | \( A_p \) (mm²) | \( P_{max} \) (kN) | \( T_{max} \) (N) | \( T_{y} \) (kN) | \( \Delta b_y \) (m) | \( \Delta \bar{b}_{br} \) (m) | \( \Delta t_{br} \) (m) |
|----------------|-------|------------|-----------------|-------------------|-----------------|---------------|----------------|----------------|----------------|
| D-BRBF         | 3     | FH3        | 750             | 506.151           | 432.608         | 298.35        | 0.006479      | 0.1334         | 0.1334         |
|                | 2     | FH2        | 1300            | 877.328           | 749.853         | 517.14        | 0.006479      | 0.1334         | 0.1334         |
|                | 1     | FH1        | 1400            | 944.815           | 807.534         | 556.92        | 0.006479      | 0.1334         | 0.1334         |
|                | 3     | FH3        | 900             | 607.381           | 519.129         | 385.02        | 0.006479      | 0.1334         | 0.1334         |
| BRBF-F         | 2     | FH2        | 1300            | 877.328           | 749.853         | 517.14        | 0.006479      | 0.1334         | 0.1334         |
|                | 1     | FH1        | 1500            | 1012.3            | 865.215         | 596.7         | 0.006479      | 0.1334         | 0.1334         |
|                | 3     | FH3        | 900             | 607.381           | 519.129         | 385.02        | 0.006479      | 0.1334         | 0.1334         |
| BRBF-P         | 2     | FH2        | 1300            | 877.328           | 749.853         | 517.14        | 0.006479      | 0.1334         | 0.1334         |
|                | 1     | FH1        | 1500            | 1012.3            | 865.215         | 596.7         | 0.006479      | 0.1334         | 0.1334         |

### 4.2. Structural performance

#### 4.2.1. Target displacement

The static non-linear second order analysis using \( P - \Delta \) effects were performed. The target displacements (Td) were computed for ultimate limit state (ULS) according to N2 method [7].

Were analyzed the demand deformations in the elements of the structure at the calculated target displacement and were compared with allowed deformations in terms of radians for plastic rotations at
the end of the beams and columns, and in terms of linear deformation at half-length of buckling restrained braces.

The plastic hinges for D-BRBF, BRBF-F and BRBF-P structures for modal distribution of forces at target displacement (Td) deformation in an upper point at level three and comparison with allowed deformations are presented in table 6 and respectively table 7 and table 8.

**Table 6.** Result of pushover analysis for D-BRBF structure.

| Target displacement -158.20 [mm] |
|----------------------------------|
| Acceptance criteria - ULS        |
| Images of model at the target displacement |
| Demand vs. Capacity in terms of plastic rotations per element type [mrad] |
| Beam                            |
| 7.09 < 84.6                     |
| Column                          |
| 5.69 < 24.93                    |
| Brace                           |
| 41.3 < 133.40                   |

**Table 7.** Result of pushover analysis for BRBF-F structure.

| Target displacement -159.46 [mm] |
|----------------------------------|
| Acceptance criteria - ULS        |
| Images of model at the target displacement |
| Demand vs. Capacity in terms of plastic rotations per element type [mrad] |
| Beam                            |
| 0 < 10.13                       |
| Column                          |
| 9.65 < 12.24                    |
| Brace                           |
| 47.3 < 133.40                   |

**Table 8.** Result of pushover analysis for BRBF-P structure.

| Target displacement -159.01 [mm] |
|----------------------------------|
| Acceptance criteria - ULS        |
| Images of model at the target displacement |
The results of pushover analysis and determination of target displacements for ultimate limit states (ULS) using N2 method for each of structures are presented in figure 5. The failure of all structures takes place along with failure of BRBs.

From pushover analysis using N2 method we can see that all structures have a good behavior and member’s deformations accomplishes the ULS deformations criteria. A better behavior has D-BRBF structure which is more resistant. BRBF-F and BRBF-P structures have nearly similar behavior, but BRBF-F structure has a better ductility –same ductility as D-BRBF. In matter of ductility D-BRBF and BRBF-F shows better behavior, the D-BRBF structure failed at top displacement d=2.5Td, BRBF-F structure failed at top displacement d=2.5Td, but BRBF-P structure failed at top displacement d=2.1Td, where Td is target displacement. In matter of base shear force resistance D-BRBF structure is more...
resistant than BRBF-F structure with 32.9%, and then BRBF-P structure with 38.8%. BRBF-F structure is more resistant than BRBF-P structure with 8.76%.

4.2.2. Analysis of over strength factor $\Omega^T$. In this chapter we analyzed the over strength factor $\Omega^T$ resulted from analyses as a ratio between maximum pushover base shear force and base shear force calculated according to lateral force method in P100/1 [5].

The design $\Omega^T$ factor is an over-strength factor used to calculate the non-dissipative components of the structure according to P100/1 [5].

The over-strength factor $\Omega^T$ for buckling restrained braces (BRB) is calculated according to paragraph 6.11.5 (1) from P100/1 [5] with relation:

$$\Omega^T = \beta \cdot \omega \cdot \gamma_{OV} \cdot \Omega^N$$ (12)

where $\Omega^N = \min \left( \Omega_i^N \right)$, and $\Omega_i^N = N_{pl,Rd,i} \cdot \left( N_{Ed,i} \right)^{-1}$; $\gamma_{OV} = 1.25$, -the over-resistance factor for S355 steel grade according to paragraph 6.2 (5) from P100/1 [5]; $\beta$, and $\omega$ were described in section 3.

The over-strength factor $\Omega^T$ for unbraced frame beams (MRF) is calculated according to paragraph 6.6.3 (1) from P100/1 [5] with relation:

$$\Omega^T = 1.1 \cdot \gamma_{OV} \cdot \Omega^M$$ (13)

where $\Omega^M = \min \left( \Omega_i^M \right)$, and $\Omega_i^M = M_{pl,Rd,i} \cdot \left( M_{Ed,i} \right)^{-1}$.

- **D-BRBF structure**
  
  From elastic calculus for D-BRBF structure was obtained $\Omega_{D-BRB}^T = 2.37$ from primary dissipative members - BRBs and $\Omega_{MRF}^T = 2.76$ from secondary dissipative members - beams of MRFs. It was adopted the most detrimental over-strength factor - $\Omega_{D-BRB}^T = \Omega_{MRF}^T = 2.76$ for checking of non-dissipative members of the D-BRBF structure.

- **BRBF-F structure**
  
  From elastic calculus for BRBF-F structure was obtained $\Omega_{BRBF-F}^T = 2.33$ from primary dissipative members – BRBs. The secondary dissipative members does not exist in this structure due to the fact that the beams of unbraced frames are pinned.

- **BRBF-P structure**
  
  From elastic calculus for BRBF-P structure was obtained $\Omega_{BRBF-P}^T = 2.25$ from primary dissipative members – BRBs. The secondary dissipative members do not exist for this structure.

The results of pushover analysis for ULS is represented in figure 6 which represents the base shear force vs. top displacement curve of structure for modal distribution of forces at ultimate limit states (ULS), considering the failure of BRB.
The Base shear force $F_b$ was computed according to lateral force method from P100/1 [5], paragraph 4.5.3.2.2.

Calculated base shear force is $F_b = 519 \, kN$ for D-BRBF, BRBF-F and BRBF-P structures.

The $\Omega$ factor is calculated with formula:

$$\Omega^T = F_{\text{max.push}} \cdot (F_b)^{-1}$$

(14)

where $F_{\text{max.push}}$ - is the maximum force from pushover curve; $F_b$ - Base shear force calculated according to paragraph 4.5.3.2.2 from P100/1 [5].

The results obtained for calculus of over strength factor of $\Omega^T$ calculated after pushover analysis, and using P100/1 code approach are presented in table 9.

|       | D-BRBF | BRBF-F | BRBF-P |
|-------|--------|--------|--------|
| Pushover |  3.31  |  2.22  |  2.03  |
| Code   |  2.76  |  2.33  |  2.25  |

The difference in percent’s between elastic and pushover analysis is: for D-BRBF – (+16.6%), for BRBF-F – (-4.72%) and for BRBF-P – (-9.77%).

The analysis showed that there are differences between code based calculation of over strength factor of $\Omega^T$ and over strength factor of $\Omega^T$ calculated after pushover analysis.

For BRBF-F and BRBF-P are relatively small differences - (-5…-10%), but for D-BRBF structure is a 17% difference. This shows that code based calculus of over strength factor $\Omega^T$ for this type of dual MRF-BRBF structure is underestimated, and this type of dual MRF+BRBF structures needs more research.
4.2.3. Analysis of forces. In this chapter we analyzed the maximum efforts (P, M, V) in elastic analysis from most loaded non-dissipative members (columns and beams of buckling restrained braced frames) from non-dissipative seismic combination and compared with efforts in pushover analysis curves at maximum force response.

- **D-BRBF structure**

The efforts from elastic and pushover analysis for D-BRBF structure is presented in table 10.

|                | P (kN) | M (kNm) | V (kN) |
|----------------|--------|---------|--------|
| Floor 1 column | Elastic analysis -1403.2 | -165.0 | -83.6  |
|                | Pushover analysis -1235.3 | -460.7 | 189.1  |
| Floor 1 BRBF beam | Elastic analysis -642.0 | -85.9 | 57.2   |
|                | Pushover analysis -635.2 | 256.5 | 133.8  |

The ratio of efforts is computed with formula:

\[
f = \frac{F_{\text{max,push}}}{(F_{\text{elastic}})}^{1}
\]

where \( F_{\text{max,push}} \) - The maximum effort from pushover analysis; \( F_{\text{elastic}} \) - The maximum effort from elastic analysis from non-dissipative seismic combination.

The results for floor 1 column are presented in figure 7 (a) and for floor 1 BRBF beam in figure 7 (b).

![](image)

**Figure 7.** The ratio of efforts \( f \) for column (a) and for BRBF beam (b) in D-BRBF structure.

- **BRBF-F structure**

The efforts from elastic and pushover analysis for BRBF-F structure is presented in table 11.

|                | P (kN) | M (kNm) | V (kN) |
|----------------|--------|---------|--------|
| Floor 1 column | Elastic analysis -1377.9 | 72.9 | 32.4   |
|                | Pushover analysis -1554.3 | -194.5 | 23.0   |
| Floor 1 BRBF beam | Elastic analysis -589.3 | -63.9 | 66.3   |
|                | Pushover analysis -670.7 | 247.6 | 132.4  |

The ratio of efforts is computed with formula (15).

The results for floor 1 column are presented in figure 8 (a) and for floor 1 BRBF beam in figure 8 (b).
The ratio of efforts $f$ for column (a) and for BRBF beam (b) in BRBF-F structure.

- **BRBF-P structure**

The efforts from elastic and pushover analysis for BRBF-F structure is presented in table 12.

**Table 12.** The efforts from elastic and pushover analysis for BRBF-P.

|                   | Floor 1 column | Floor 1 BRBF beam |
|-------------------|----------------|-------------------|
|                   | Elastic analysis | Pushover analysis | Elastic analysis | Pushover analysis |
| $P$ (kN)          | -1347.3        | -1475.8           | -597.4           | -679.3           |
| $M$ (kNm)         | -21.0          | -167.0            | -58.2            | 250.4            |
| $V$ (kN)          | 6.0            | -34.2             | 63.5             | 133.5            |

The ratio of efforts is computed with formula (15).

From this analysis we can conclude that differences between efforts of non-dissipative members from elastic analysis and efforts from pushover analysis at maximum failure force for the same non-dissipative members are quite different. For D-BRBF structure the differences of axial forces ($P$) are nearly the similar, most differences are for bending moment ($M$) - $\Omega^T_M = 2.79$ for column and $\Omega^T_M = 2.99$ for beam, and for shear force ($V$) - $\Omega^T_V = 2.26$ for column and $\Omega^T_V = 2.34$ for beam.
For BRBF-F structure the differences of axial forces (P) are nearly the similar, most differences are for bending moment (M) - $\Delta M = 2.67$ for column and $\Delta M = 3.88$ for beam, and for shear force (V) - $\Delta V = 0.71$ for column and $\Delta V = 2.0$ for beam.

For BRBF-P structure the differences of axial forces (P) are nearly the similar, most differences are for bending moment (M) - $\Delta M = 7.94$ for column and $\Delta M = 4.31$ for beam, and for shear force (V) - $\Delta V = 5.69$ for column and $\Delta V = 2.10$ for beam.

The differences for axial force for D-BRBF structure are:
- Axial force: (-12.0 %) for column, and (-1.1 %) for beam. Bending moment and shear force have big differences.

For BRBF-F structure: Axial force- 11.3 % for column, and 12.1 % for beam. Bending moment and shear force have big differences except shear force for column which is (-29.1 %).

For BRBF-P structure: Axial force- 8.7 % for column, and 12.0 % for beam. Bending moment and shear force have big differences.

We can observe that efforts that varies the most are bending moment (M) and shear force (V) for all structures, but axial forces (P) remain quite similar. The BRBF-P structure has the most important differences of M and V efforts.

5. Conclusions

The results show that in matter of resistance the D-BRBF structure is more resistant. In matter of ductility, - the D-BRBF and BRBF-F structures has better ductility than BRBF-P structure. The total weight of frames is 30.67 t for D-BRBF, 29.75 t for BRBF-F and 30.73 t for BRBF-P structure, this shows that dual MRF+BRBF structure has nearly the same weight as BRBF structure, but instead is more resistant.

The pushover analysis according to N2 method showed that structures fulfils all deformations requirements at target displacement for ULS. The analysis of over strength factor $\Omega_T$ as a ratio between maximum pushover base shear force and base shear force calculated according to lateral force method in P100/1 showed a relatively small difference (~(-5…-10 %) for BRBF-F and BRBF-P structures, but a 17 % difference for D-BRBF structure. This shows that code based calculus of over strength factor $\Omega_T$ for this type of dual MRF-BRBF structure is underestimated, and needs more research.

The analysis of forces in non-dissipative members shows that the code approach provides a reasonable approximation for axial forces, but generally underestimate by large amount the bending moment and shear force demands. Consequently, as observed from pushover analysis, the resistance of non-dissipative members is insufficient for preventing their plastic hinging. Improved design approach is necessary, and will be addressed in future studies.

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