A Comparative Study on FBD and DBD Methodology with Emphasis on Bridges with Seismic Irregularity

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Abstract. One of the major developments in seismic design over the past few decades is the increased emphasis for limit states design now generally termed as Performance Based Engineering. Performance Based Seismic Design (PBSD) uses Displacement Based Design (DBD) methodology where-in structures are designed for a target level of displacement rather than Force Based Design (FBD) methodology, where-in force or strength aspect is used. Indian codes still follow FBD methodology compared to other modern codes like CalTrans, which follow DBD methodology. Hence in the present study, a detailed review of the two most common design methodologies i.e. FBD and DBD will be discussed, along with its comparison on bridge models with and without ‘seismic regularity’. Bridges with irregular column height are studied to discuss the inherent discrepancy associated with FBD while dealing with ‘seismic irregularity’.

1. Introduction

Design for seismic resistance has been undergoing a critical reappraisal in recent years, with the emphasis changing from “strength” to “performance” [6]. Previously they were believed to be synonymous, i.e. an increase in strength will result in an increase in performance in terms of safety and damage reduction. Also, it is now a proven fact that, inelastic characteristics can be seldom described through elastic idealisation. But, still Indian Standards like IRC 112 (2011) follows FBD methodology in comparison to modern codes like CalTrans (2013), which follows DBD methodology. Among the various DBD procedures, the one established by Priestley et al. (2007) i.e., DDBD, is the most preferred among the researchers. Hence, in the present study a detailed review of both FBD and DDBD methodologies are discussed with design illustrations for better understanding the significance of the latter. Also, the study evaluates the performance level achieved by structures designed as per both design methodologies during seismic excitation.

Unequal column height is one of the main irregularities seen in bridges, particularly while negotiating steep valleys but, IRC 112 (2011) does not provide any specific design recommendations for these bridges. Thus, it is inevitable to assess the performance level achieved by the irregular bridges during seismic excitation as irregularities make bridges vulnerable to seismic damage [7].
2. Details of Modelling and Analysis

For bridges, it is generally assumed that, the deck or the superstructure portion behaves elastic and the substructure portion mainly columns, act as ductile members during seismic excitations. Thus, the lateral load resisting members are the columns and the performance of the entire structure depends on their performance. So, in the present study columns are designed for the two design methodologies while deck remains the same, which is designed as per FBD. Three dimensional (3D) finite element model of bridges are done in SAP2000 NL to assess its global seismic performance through pushover analysis. Three span PSC box-girder bridges with individual span length of 30 m are used in all the cases studied. The geometry and dimension detail of the twin celled box-girder deck used in the present study is shown in figure 1. The deck is cast monolithic with the columns at bents, and at abutments it is provided with sliders permitting translational motion along the longitudinal direction similar to that of a semi-integral bridge. Super Imposed Dead Load (SIDL) of 2 kN/m is provided for considering the mass of wearing course and other secondary elements which does not contribute additional stiffness to the structure. The bridge models considered are of single column bents and the columns at both bents are of the same cross-sectional dimension 1.5 m × 3 m. The height of column at Bent-1 (B-1) is varied relatively from 0.5, 0.65, 0.8 and 1 with respect to the column at Bent-2 (B-2), which is of 16 m height, as shown in figure 2. Hence, the column at B-1 will have heights of 8 m, 10.4 m, 12.8 m and 16 m, respectively for Relative Height (RH) ratio of 0.5, 0.65, 0.8 and 1. M40 concrete and Fe 415 steel is used in all the models considered.

3. Comparison of DDBD and FBD Methodology

Although the current force-based design method is considerably improved compared to the procedures used earlier, there still exist certain fundamental problems with this procedure when applied to reinforced concrete or reinforced masonry structures. After the Loma Prieta earthquake in 1989, extensive research has been conducted to develop improved seismic design criteria for bridges, emphasizing the use of displacements rather than forces as a measure of earthquake demand and damage in the structures [4][8][9]. Extensive work on the application of capacity design principles to assure ductile mechanisms and concentration of damage in specified regions has also been conducted.
Several DBD methodologies have been proposed, among them, the Direct Displacement-Based Design Method (Priestley, 1993) has proven to be effective for performance-based seismic design of bridges, buildings and other types of structures [10][11][12][13][14].

3.1. Discussion on FBD and DDBD procedure
The sequence of operation required for force-based design of a structure is summarised in the flow chart shown in figure 3.

![Flow chart of force-based design methodology.](image)

In this method of design, elastic stiffness of the member is used in the estimation of total base shear demand in the structure. The total base shear demand, $V_b$, is calculated as, $V_b = A_h \times W$; where, $A_h$ is the seismic acceleration coefficient calculated from the elastic acceleration response spectrum and $W$ is the seismic weight of the structure. $V_b$ is then distributed to the individual columns according to their initial

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Figure 3. Flow chart of force-based design methodology.

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stiffness. Hence, the distributed shear will be proportional to $1/H^3_i$, where, $H_i$ is the height of the corresponding column. Considering the ductility, overstrength and redundancy of the column, the code recommends a Reduction factor ($R$), for the columns. Thus, the in-elastic base shear demand on columns $V_{inelastic}$ is calculated as $V_{elastic}/R$. This base shear value is used in the estimation of $M_{design}$. To check the safety condition, the moment capacity, $M_{capacity}$, is estimated through Moment-Curvature analysis and $V_{capacity}$ based on empirical relations given in the code. The section is considered safe, if both moment and shear demand values are less than their corresponding capacity values. Here, the elastic base shear estimated is used instead of $V_{inelastic}$. Finally, the deflection ($\delta$) of the structure (column top displacement for bridges) is checked against the maximum permissible limits stipulated by the codes, to ensure whether it is serviceable under the selected lateral loading condition.

**Figure 4.** Flow chart of direct displacement-based design methodology.
Displacement-Based techniques gained momentum with the knowledge of the discrepancies associated with the Force-Based Design method. Consequently, a number of new design methods or improvements to the existing methods are evolved. The two main methodologies associated with Deformation-Specification Based Design are Equal Displacement-Based Design, which uses the pre-yield elastic stiffness as in conventional FBD and hence requires a number of iterations; and DDBD method which utilises the secant stiffness corresponding to the maximum displacement and hence need little or no iterations to design a structure corresponding to a specified displacement limit. Thus, DDBD is the most preferred technique among the several displacement-based methods owing to its simplicity in application and wider applicability to several structures.

DDBD is a design procedure for determining the required strength of different structural systems to ensure that, a given performance state defined by flexural strain or drift limits is achieved, under a specified level of seismic intensity. From this design strength, the required moment capacity at intended locations of plastic hinges or shear capacity of seismic isolation devices for seismic isolated structures can be determined.

The procedure for DDBD is portrayed in figure 4, where-in a target displacement and the effective stiffness of the structure have to be determined in order to calculate the base shear demand. The procedure described in Priestley et al. (2007) has been adopted in this study. Here, the structure is designed for a target displacement \( \Delta_d(\Delta_d = \Delta_s \mu) \); where, \( \Delta_s \) is the yield displacement which is based on the section size and \( \mu \) is the displacement ductility. The target displacement value corresponding to the ‘damage control’ limit state is used in the design. Based on the ductility, equivalent viscous damping of the system, \( \xi \) is estimated. Using the displacement response spectra generated for the estimated level of damping and target displacement, effective time period \( (T_e) \) of the structure is calculated. The \( T_e \) obtained is used for the estimation of the effective stiffness \( (K_e) \) of the structure. If the effective stiffness estimated is less than the initial elastic stiffness of the structure \( (K_{\text{elastic}}) \), the procedure is continued else the section size is increased. The total base shear, \( V_d \) is calculated as, \( V_d = K_e \times \Delta_d \), which is then distributed to the individual columns in inverse proportion to their height. Finally, the column is checked for moment and shear values as portrayed in Figure 4. In this method, the design moment \( M_{design} \) for all the columns remain the same as the \( V_d \) is distributed in inverse proportion to the column height.

3.2. Discrepancies Associated with FBD
In FBD, constant member stiffness independent of the member strength is assumed, which implies that, the yield curvature is directly proportional to strength, but in reality, the yield curvature is independent of member strength and the stiffness is directly proportional to strength and hence, the assumption of constant member stiffness is no longer valid. As a consequence of this invalid assumption, successive iteration must be carried out before an adequate elastic characterization of the structure is obtained, which is rarely performed by the designers. Also, the member stiffness may be based on gross-section stiffness or sometimes a reduced stiffness to represent the influence of cracking. And, clearly the stiffness values assumed affects the design seismic forces selected from the elastic response spectrum. The basic assumption associated with FBD is that the elastic characteristic of the structure is the best indicator of its inelastic performance. But it is a well-known fact that, beyond yield the initial elastic stiffness is no longer valid due to; the crushing of concrete, Bauschinger softening of reinforcing steel and damage on crack surfaces. This gives the impression that, structural characteristics that represent the performance at maximum response might be better predictors of performance than initial values of stiffness and damping.

The force reduction factor calculated based on the ductility demand following an equal-displacement approximation involves many uncertainties. It is a known fact that, the equal -displacement approximation is inappropriate to very short period and very long period structures and its validity on medium period structures is also doubtful when the hysteretic character of the inelastic system deviates significantly from elasto-plastic behaviour [15]. Also, there exists a significant difference in the criteria by which yield and ultimate displacement values are chosen in different countries, leading to significantly varied values of force reduction factors used in the codes of those countries (e.g. America, Japan, India etc.). This gives an impression that the absolute value of the member strength is of relatively minor importance. A key tenet of FBD is the usage of unique ductility capacity and hence, unique force reduction factor for different structural systems. In the case of a bridge, the column/pier displacement
ductility ($\mu_d$) and hence, force reduction factor is found to be dependent on its height as evident from (1). Thus, using the same response reduction factor irrespective of the ductility of the structural system is absurd [15].

$$\mu_d = 1 + 3 \frac{\varphi_p L_p}{\varphi_y H}$$  \hfill (1)

where, $\varphi_p =$ Idealised plastic curvature capacity, $\varphi_y =$ yield curvature, $L_p =$ Plastic Hinge Length.

Following an equal displacement approximation in FBD, it is assumed that, as the strength of the structure increases by reducing the force-reduction factor, its safety increases. This is based on the assumption that, the members will have constant stiffness independent of strength, which is proven to be wrong in the discussion above. By conducting numerical experiments on bridge piers with varied percentage of longitudinal reinforcements Priestley et al. (2007) found that, the strength and stiffness of the member increases with increase in reinforcement ratio while its ductility capacity and displacement capacity reduces. But, as the member strength increases its stiffness also increases which leads to a reduction in its elastic period and hence, a reduced displacement demand from the displacement demand spectra. Thus, it can be summarised based on their study that, the reduction in displacement demand to capacity ratio with increase in member strength is negligibly low, which proves that the argument of safety enhancement with increase in member strength is invalid.

Another serious issue with FBD is its application on structures with dual load path, like that of a bridge, where its superstructure is designed to behave elastically while its columns/piers are designed to have in-elastic response upon seismic action. This is particularly important while conducting transverse seismic analysis of bridges. In this case, assuming force-reduction factor for piers/columns may adversely affect the elastic behaviour assumed in its superstructure design.

3.3. Design of Bridges based on FBD and DDBD

The bridge models considered for the comparison of DDBD and FBD methodologies include both regular and irregular bridges. Irregularity in seismic action is brought by providing bridge columns of unequal height along the length of the bridge. Here, the longitudinal and confinement reinforcement of bridge columns are provided in accordance to the following three conditions; (a) based on the actual moment and shear values obtained from the analysis, (b) satisfying the minimum reinforcement requirements as in IRC 112 and CalTrans (2013) for FBD and DDBD, respectively, and (c) modified minimum longitudinal reinforcement ratio for FBD in accordance with AASHTO (2012), while keeping all other structural characteristics the same as discussed in section 2. A brief discussion on the analysis and design of bridge with RH = 0.5 is provided below for both DDBD and FBD for better clarity.

3.3.1. Illustration of FBD. In FBD, the design base shear demand ($V_d$) is estimated based on the seismic acceleration coefficient ($A_h$) obtained from the response spectrum (see Figure 5) and the seismic weight ($W$) of the bridge. The bridge is assumed to be situated in Zone V and having an Importance factor ($I$) of 1, corresponding to ‘Normal’ type of bridges as per IRC 6 (2010). The elastic time period ($T$) of the bridge model considered is calculated based on modal analysis, which is found to be 0.406 s and the initial elastic stiffness can be estimated based on the expression given in (2), which is estimated as 409066.7 kN/m.

$$K_{\text{elastic}} = \frac{V_d}{\Delta_{\text{max}}},$$  \hfill (2)

where, $V_d = A_h \times W$ and $\Delta_{\text{max,elastic}} = \frac{T^2}{4\pi^2} S_a g$.

The elastic base shear demand ($V_{\text{elastic}}$) is found to be 15078.2 kN after using a load factor of 1.5 and the in-elastic base shear demand ($V_{\text{inelastic}}$) after applying a reduction factor of 3 as per IRC 6 (Amendment: August 2014) is estimated to be 5026.1 kN. The total elastic base shear demand ($V_{\text{elastic}}$) is distributed to the individual columns based on their initial stiffness i.e. the base shear distribution is inversely proportional to the cube of their corresponding heights as shown in (3). This results in a base shear value of 13402.8 kN in the short column at B-1 and 1675.4 kN in the long column at B-2. The design moment ($M_{\text{design}}$) is calculated from inelastic base shear demand ($V_{\text{inelastic}}$) and is found to be 17870.4 kNm and 4468 kNm in the columns at B-1 and B-2, respectively. Here, the design moment will
be distributed to the individual columns as inverse proportion to the square of their column heights. Table 1 gives the value of similar design parameters for the bridge models with other RH ratios considered in the present study. The variation in design base shear and moment is portrayed in Figure 7 and 8, respectively, and the corresponding reinforcement is given in Table 3, 4 and 5, for all the three cases discussed above.

\[
V_i = \frac{1}{\Sigma \frac{1}{H_i^3}} V_{\text{elastic}}
\]

(3)

Table 1. Design parameters for FBD.

| RH | T (s) | \(K_i\) (kN/m) | \(V_d\) (kN) | \(V_{\text{inelastic}}\) (kN) |
|----|------|--------------|-------------|------------------|
| 1  | 0.827| 102551.0     | 10289.8     | 3429.9           |
| 0.8| 0.686| 146731.0     | 12168.8     | 4056.3           |
| 0.65| 0.552| 223942.2     | 15260.4     | 5086.8           |

Figure 5. Response spectrum as per IRC 6 (2010).

3.3.2. Illustration of DDBD. In the selected bridge model with RH of 0.5, the column height at B-1 is 8 m while at B-2 it is 16 m. Since, column at B-1 is the shortest, the yield and target displacement will be governed by this column. Confinement reinforcement of 12 mm dia. @ 200 mm c/c connecting each layer of longitudinal bar is provided, so that, for a longitudinal reinforcement percentage of I, the confinement reinforcement percentage is 0.565, which is within the minimum specified value as per CalTrans (2013), for a Peak Ground Acceleration (PGA) equivalent to Zone V of IRC 6 (2010). According to Montes and Aschleim (2003) the effective yield curvature for a member section with Fe 400 grade steel is as given in (4), as per moment-curvature analysis. And, Priestley et al. (2007) advocates a rotation (\(\theta\)) value of 0.01 for achieving ‘damage control’ limit state. At this limiting state non-structural damage is least expected. The ‘damage control’ limit state can also be defined based on the compressive strain limit of confined concrete as given in (5). Based on (5) and using the expression for yield displacement as in (6), the yield and target displacement corresponding to ‘damage control’ limit state are found to be 36.57 mm and 120 mm, respectively. Hence, the ductility of columns at B-1 and B-2 are 3.28 and 0.82, respectively. Now, the equivalent viscous damping ratio of individual column can be estimated using (7), where the first part of the equation is for 5% material damping of concrete and the second part is for hysteresis damping, calculated from ductility [15]. The equivalent viscous damping of columns at B-1 and B-2 are found to be 0.148 and 0.05, respectively. The equivalent viscous damping for the whole system can be estimated using (8). In DDBD the base shear is distributed as
inverse proportion to the column height and hence, the equivalent viscous damping of the whole system is found to be 11.53%. The spectral reduction factor corresponding to the equivalent system damping can be calculated from (9).

\[
\varphi_y = 2.4 \times \frac{\varepsilon_{ys}}{d} 
\]

\[
\varepsilon_{cm} = 0.004 + 1.4\rho_s f_{yh} \frac{\varepsilon_{su}}{f_{cc}} 
\]

\[
\Delta_y = \frac{\varphi_y H^2}{6} 
\]

\[
\xi_{eq} = 0.05 + 0.444 \frac{(\mu - 1)}{\pi \mu} 
\]

\[
\xi_{sys} = \frac{\sum m V_i \xi_i}{\sum m V_i} 
\]

\[
R_{\xi} = \left( \frac{0.07}{0.02 + \xi_{sys}} \right)^{0.5} 
\]

where, \(\varphi_y\) = yield curvature; \(\varepsilon_{ys}\) = yield strain of longitudinal reinforcement; \(d\) = effective depth of the section; \(\varepsilon_{cm}\) = limiting compressive strain of confined concrete; \(\rho_s\) = volumetric ratio of confinement reinforcement; \(f_{yh}\) = yield strength of confinement; \(\varepsilon_{su}\) = strain at maximum stress; \(f_{cc}\) = compressive strength of confined concrete [18]; \(\Delta_y\) = yield displacement; \(H\) = effective height of column; \(\xi_{eq}\) = equivalent viscous damping ratio and \(\mu\) = displacement ductility; \(V_i\) = base shear in each column.

The demand displacement response spectra based on the Indian code, IRC 6 (2010) is shown in figure 6. From the displacement response spectra the effective time period \((T_e)\) of the structure can be found out for the corresponding target displacement calculated, the value of which is found to be 1.38 s. Knowing the effective seismic weight \((W_e)\) of the bridge the effective stiffness \((K_e)\) can be calculated using the expression given in (10). The value of \(W_e\) for the bridge model selected is 22338 kN and the corresponding value of \(K_e\) is found to be 47203.8 kN/m.

\[
K_e = \frac{4\pi^2 W_e}{g T_e^2} 
\]

Figure 6. Displacement response spectra.

The total base shear demand \(V_d\), which is the product of effective stiffness and target displacement is estimated to be 5664.5 kN. This base shear is distributed to each individual column in inverse proportion to their height. Hence, the base shear in each individual column \((V_i)\) can be estimated using the expression given in (11) and is found to be 3776.3 kN and 1888.2 kN, respectively at B-1 and B-2. This method of base shear distribution in DDBD leads to equal bending moment, and hence equal longitudinal reinforcement in columns at both bents. Table 2 gives the value of similar design parameters for the
bridge models with other RH ratios considered in the present study. The variation in design base shear and moment is portrayed in Figure 7 and 8, respectively, and the corresponding reinforcement is given in Table 3, 4 and 5 for all the three cases discussed.

\[ V_i = \frac{1}{H_i} \sum \frac{1}{H_i} V_d \]  

(11)

The base shear at B-1 is found to be increasing in Figure 7 as the irregularity in bridges increases, while at B-2 it gets decreased in the case of FBD. But, for the bridges designed as per DDBD methodology, the base shear remains comparatively similar and well below the corresponding value at RH = 1. This clearly indicates the relevance of DDBD over FBD methodology. As discussed earlier, the variation in base shear value points out that, in FBD short columns determine the failure criteria, whereas in DDBD a more regularity in seismic performance can be achieved.

Table 2. Design parameters for DDBD.

| RH  | \( \Delta_y \) (mm) | \( \Delta_d \) (mm) | \( \mu_1 \) | \( \mu_2 \) | \( \xi_1 \) | \( \xi_2 \) | \( \xi_{sys} \) | \( R_e \) (s) | \( T_e \) (s) | \( W_e \) (kN) | \( K_e \) (kN/m) | \( V_d \) (kN) |
|-----|------------------|------------------|----------|----------|-------|-------|--------|---------|---------|-------------|--------------|-------------|
| 1   | 146.30           | 240.00           | 1.64     | 1.64     | 0.105 | 0.105 | 0.105  | 0.748   | 1.72    | 23238       | 31610.6      | 7586.5      |
| 0.8 | 93.63            | 192.00           | 2.05     | 2.05     | 0.122 | 0.084 | 0.105  | 0.748   | 1.72    | 22878       | 31120.9      | 5975.2      |
| 0.65| 61.81            | 156.00           | 2.52     | 1.07     | 0.135 | 0.059 | 0.105  | 0.748   | 1.72    | 22608       | 30753.6      | 4797.6      |

Figure 7. Variation of base shear at bents with design methodology.

In the case of moment variation at bents, a similar trend as that of the base shear value is obtained in case of FBD methodology. This means a higher amount of steel reinforcement at shorter columns and a comparatively lower quantity at longer columns. It is seen that, the moment value at B-1 for an RH of 0.5 is lesser than that of RH of 0.65 in case of bridges designed as per FBD. This is due to the fact that, the spectral acceleration value corresponding to the elastic time period obtained from the elastic response spectrum for the bridges with both RH values remain same i.e. \( Sa/g = 2.5 \) (maximum value). But, an increased column height at B-1 for bridge with RH = 0.65 increases its seismic weight, and hence the total base shear for bridge with RH = 0.65 remains higher than that of bridge with RH = 0.5. This results in a higher moment at B-1 for bridge with RH = 0.65, as design moment is estimated as the product of distributed inelastic base shear and half the column height.
Figure 8. Variation of moment at bents with design methodology.

As per IRC 112, the minimum longitudinal reinforcement required is 0.2% $A_c$ or 0.1 $N_{Ed}/f_{yd}$, where $A_c$ is the gross cross-sectional area of concrete, $N_{Ed}$ is the design axial compression force and $f_{yd}$ is the design yield strength of the reinforcement; and the minimum transverse reinforcement volumetric ratio for M40 concrete and Fe 415 steel is 0.593%, with a maximum spacing of 200 mm. As per CalTrans (2013), the minimum longitudinal reinforcement ratio is 1% of gross sectional area and the maximum allowable spacing of confinement reinforcement is 200 mm or six times the nominal diameter of longitudinal rebar, whichever is less.

Table 3, 4 and 5 shows the reinforcement required as per the three cases discussed in section 3.3. The minimum longitudinal reinforcement percentage specified for FBD as per IRC 112 is very small compared to CalTrans. For a column section size of 1.5 m × 3 m, the minimum reinforcement required is only 9000 mm$^2$, which means nearly 30 numbers of 20 mm diameter bars in total. This is proven to be insufficient for achieving ‘life safety’ level of performance at MCE level of seismic hazard (refer table 7). Also, the performance level of columns at both bents varies considerably even with slight reduction in RH ratio. Hence, based on the present study, the minimum percentage of longitudinal reinforcement in IRC 112 following FBD methodology is recommended to be modified to 1% as in AASHTO (2012), ACI 318 (2008) and CalTrans (2013). The studies conducted by Halvorsen (1987) further proved that, column with longitudinal rebar percentage less than 1% has not exhibited sufficient ductility. The minimum confinement reinforcement specified by IRC 112 is found to be higher than CalTrans, which is better from ductility point of view.

Table 3. Reinforcement at columns of bridges designed with actual moment and shear values.

| RH  | Design Method | Bent-1 Longitudinal Reinforcement | Bent-1 Transverse Reinforcement | Bent-2 Longitudinal Reinforcement | Bent-2 Transverse Reinforcement |
|-----|---------------|-----------------------------------|---------------------------------|-----------------------------------|---------------------------------|
|     |               | 52-25 mm φ | 12 mm φ @ 172 mm c/c | 52-25 mm φ | 12 mm φ @ 172 mm c/c |
| 1   | FBD           | 108-25 mm φ | 12 mm φ @ 200 mm c/c | 108-25 mm φ | 12 mm φ @ 200 mm c/c |
|     | DDBD          | 64-25 mm φ | 12 mm φ @ 110 mm c/c | 42-25 mm φ | 12 mm φ @ 196 mm c/c |
| 0.8 | FBD           | 78-25 mm φ | 12 mm φ @ 200 mm c/c | 78-25 mm φ | 12 mm φ @ 200 mm c/c |
|     | DDBD          | 76-25 mm φ | 12 mm φ @ 78 mm c/c | 34-25 mm φ | 12 mm φ @ 231 mm c/c |
| 0.65| FBD           | 58-25 mm φ | 12 mm φ @ 200 mm c/c | 58-25 mm φ | 12 mm φ @ 200 mm c/c |
|     | DDBD          | 67-25 mm φ | 12 mm φ @ 59 mm c/c | 28-20 mm φ | 12 mm φ @ 673 mm c/c |
| 0.5 | FBD           | 58-25 mm φ | 12 mm φ @ 200 mm c/c | 58-25 mm φ | 12 mm φ @ 200 mm c/c |

Note: φ denotes diameter of rebar.
Table 4. Reinforcement at columns of bridges designed as per DDBD and FBD.

| RH  | Design Method | Bent-1 | Bent-2 |
|-----|---------------|--------|--------|
|     | Longitudinal  | Transverse | Longitudinal  | Transverse |
|     | Reinforcement | Reinforcement | Reinforcement | Reinforcement |
| 1   | FBD 52-25 mm φ | 12 mm φ @ 118 mm c/c | 52-25 mm φ | 12 mm φ @ 118 mm c/c |
| DDBD 108-25 mm φ | 12 mm φ @ 200 mm c/c | 108-25 mm φ | 12 mm φ @ 200 mm c/c |
| 0.8 | FBD 64-25 mm φ | 12 mm φ @ 110 mm c/c | 42-25 mm φ | 12 mm φ @ 95 mm c/c |
| DDBD 92-25 mm φ* | 12 mm φ @ 200 mm c/c | 92-25 mm φ* | 12 mm φ @ 200 mm c/c |
| 0.65 | FBD 76-25 mm φ | 12 mm φ @ 78 mm c/c | 34-25 mm φ | 12 mm φ @ 77 mm c/c |
| DDBD 92-25 mm φ* | 12 mm φ @ 200 mm c/c | 92-25 mm φ* | 12 mm φ @ 200 mm c/c |
| 0.5 | FBD 67-25 mm φ | 12 mm φ @ 59 mm c/c | 30-20 mm φ | 12 mm φ @ 68 mm c/c |
| DDBD 92-25 mm φ* | 12 mm φ @ 200 mm c/c | 92-25 mm φ* | 12 mm φ @ 200 mm c/c |

Note: * denotes minimum value required as per IRC 112 for FBD and CalTrans (2013) for DDBD.

Table 5. Reinforcement at columns of bridges designed with revised minimum $P_t$ for FBD.

| RH  | Design Method | Bent-1 | Bent-2 |
|-----|---------------|--------|--------|
|     | Longitudinal  | Transverse | Longitudinal  | Transverse |
|     | Reinforcement | Reinforcement | Reinforcement | Reinforcement |
| 1   | FBD 92-25 mm φ* | 12 mm φ @ 100 mm c/c | 92-25 mm φ* | 12 mm φ @ 100 mm c/c |
| DDBD 108-25 mm φ | 12 mm φ @ 200 mm c/c | 108-25 mm φ | 12 mm φ @ 200 mm c/c |
| 0.8 | FBD 92-25 mm φ* | 12 mm φ @ 100 mm c/c | 92-25 mm φ* | 12 mm φ @ 100 mm c/c |
| DDBD 92-25 mm φ* | 12 mm φ @ 200 mm c/c | 92-25 mm φ* | 12 mm φ @ 200 mm c/c |
| 0.65 | FBD 92-25 mm φ* | 12 mm φ @ 100 mm c/c | 92-25 mm φ* | 12 mm φ @ 100 mm c/c |
| DDBD 92-25 mm φ* | 12 mm φ @ 200 mm c/c | 92-25 mm φ* | 12 mm φ @ 200 mm c/c |
| 0.5 | FBD 92-25 mm φ* | 12 mm φ @ 88 mm c/c | 92-25 mm φ* | 12 mm φ @ 100 mm c/c |
| DDBD 92-25 mm φ* | 12 mm φ @ 200 mm c/c | 92-25 mm φ* | 12 mm φ @ 200 mm c/c |

Table 6, 7 and 8 describes the performance level attained by the bridges at DBE and MCE level of seismic hazard for the three cases studied. It is clear that, a minimum $P_t$ of 1% makes the FBD bridges having a similar performance as that of the DDBD bridges in the present study, and hence, achieving a ‘damage control’ level of performance. Figure 9 depicts the hinge formation at the bridge columns designed as per DDBD with RH values of 1 and 0.5.

Table 6. Performance levels of bridges designed with actual moment and shear values.

| RH  | Design Method | Performance Levels |
|-----|---------------|-------------------|
|     | Bent-1 | Bent-2 |
|     | DBE | MCE | DBE | MCE |
| 1   | FBD | O | C | O | C |
| DDBD | O | IO | O | IO |
| 0.8 | FBD | O | CP | O | LS |
| DDBD | O | IO | O | IO |
| 0.65 | FBD | O | CP | O | O |
| DDBD | O | LS | O | O |
| 0.5 | FBD | O | CP | O | O |
| DDBD | O | LS | O | O |
Table 7. Performance levels of bridges designed as per DDBD and FBD.

| RH  | Design Method | Bent-1 | Bent-2 |
|-----|---------------|--------|--------|
|     |               | DBE    | MCE    | DBE    | MCE    |
| 1   | FBD           | O      | C      | O      | C      |
|     | DDBD          | O      | IO     | O      | IO     |
| 0.8 | FBD           | O      | CP     | O      | LS     |
|     | DDBD          | O      | IO     | O      | IO     |
| 0.65| FBD           | O      | CP     | O      | O      |
|     | DDBD          | O      | LS     | O      | O      |
| 0.5 | FBD           | O      | CP     | O      | O      |
|     | DDBD          | O      | LS     | O      | O      |

Table 8. Performance levels of bridges with revised minimum $P_t$ for FBD.

| RH  | Design Method | Bent-1 | Bent-2 |
|-----|---------------|--------|--------|
|     |               | DBE    | MCE    | DBE    | MCE    |
| 1   | FBD           | O      | IO     | O      | IO     |
|     | DDBD          | O      | IO     | O      | IO     |
| 0.8 | FBD           | O      | IO     | O      | IO     |
|     | DDBD          | O      | IO     | O      | IO     |
| 0.65| FBD           | O      | IO     | O      | O      |
|     | DDBD          | O      | LS     | O      | O      |
| 0.5 | FBD           | O      | LS     | O      | O      |
|     | DDBD          | O      | LS     | O      | O      |

Figure 9. Hinge formation in DDBD bridge columns-POA: (a) RH = 1, (b) RH = 0.5.

Note: O, IO, LS, CP and C represents respectively, the Operational, Immediate Occupancy, Life Safety, Collapse Prevention and Collapse level of performance.

4. Conclusions

The inherent discrepancies associated with the fundamental assumptions of FBD methodology makes it least preferred among the research community especially while dealing with seismic loads. The modern design codes are moving towards displacement-based design or rather DDBD methodology for achieving the performance objectives targeted. The present study gives a detailed review of the two most
commonly adopted design methodologies (FBD and DDBD), along with their comparison based on the performance evaluated on bridge models with and without seismic regularity. Pushover analysis using SAP2000 NL is adopted for the seismic performance evaluation of the bridge models studied. Seismic irregularity is achieved by providing columns of unequal height along the length of the bridge with relative height ratio varying from 1, 0.8, 0.65 and 0.5.

In the present study it is found that, the column irregularity in bridges will impart uneven seismic response in bridges. The shorter columns dictate the seismic performance of such bridges, particularly while adopting FBD methodology. The comparative study on FBD and DDBD methodology reveals the inherent discrepancies associated with FBD compared to DDBD, particularly for bridges with irregular column height.

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