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Can irregular bridges designed as per the Indian standards achieve seismic regularity?

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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 wherein structures are designed for a target level of displacement rather than Force Based Design (FBD) methodology where force or strength aspect is being 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 is presented. A critical evaluation of both these methodologies by comparing the seismic performance of bridge models designed using them highlight the importance of adopting DBD techniques in Indian Standards also. The inherent discrepancy associated with FBD in achieving 'seismic regularity' is highlighted by assessing the seismic performance of bridges with varied relative height ratios. The study also encompasses a brief comparison of the seismic design and detailing provisions of IRC 112 (2011), IRC 21 (2000), AASHTO LRFD (2012) and CalTrans (2013) to evaluate the discrepancies on the same in the Indian Standards. Based on the seismic performance evaluation and literature review a need for increasing the minimum longitudinal reinforcement percentage stipulated by IRC 112 (2011) for bridge columns is found necessary.

Keywords: displacement based design; force based design; performance-based design; seismic regularity; seismic design and detailing

1. Introduction

Design for seismic resistance has been undergoing a critical reappraisal in recent years, with the emphasis changing from "strength" to "performance" (Priestley 2000). Previously they are 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

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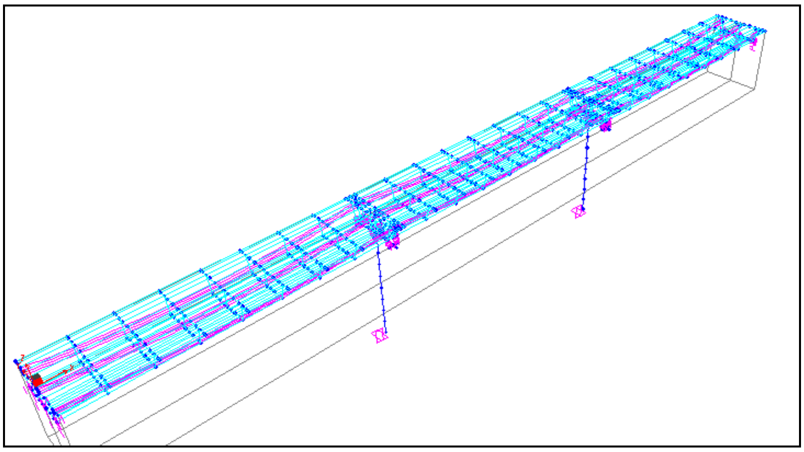


Fig. 1 Three dimensional view of the bridge model (SAP2000)

Calvi *et al.* (2007) i.e., Direct Displacement Based Design (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 (Akbari 2010, Ishac and Mehanny 2016, Tamanani, Gian *et al*. 2016). The considerable difference in minimum longitudinal reinforcement percentage for columns stipulated in Indian codes compared to other International Standards is also highlighted in view of column irregularity, loading, creep and shrinkage aspects. The bounds specified through seismic detailing provisions often regulate the design standards in achieving the level of safety and serviceability targeted. Hence, a comparison between the seismic detailing provisions of Indian Standards (IRC 112, IRC 21) to International Codes such as AASHTO (2012) and CalTrans (2013) is also discussed to assess the discrepancies in the Indian Standards.

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 i.e., mainly the columns act as a ductile member 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 bridges modelled in SAP2000 NL (refer Fig. 1) are used in assessing their 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 Fig. 2.

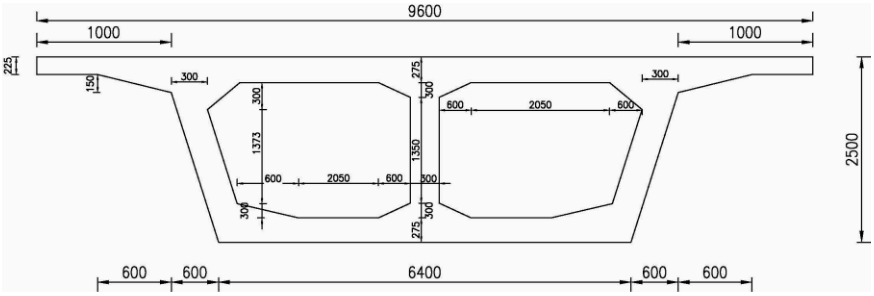
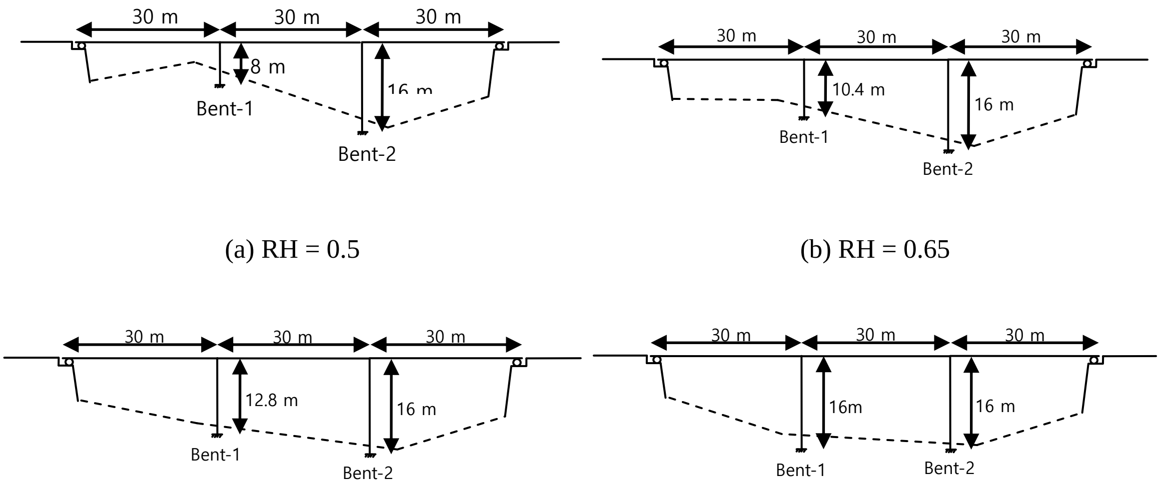


Fig. 2 Geometry and dimension detail of the deck (all dimensions in mm)



(a) RH = 0.5

(b) RH = 0.65

(c) RH = 0.8

(d) RH = 1

Fig. 3 Bridge geometry used in the study of seismic irregularity

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 Fig. 3. 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

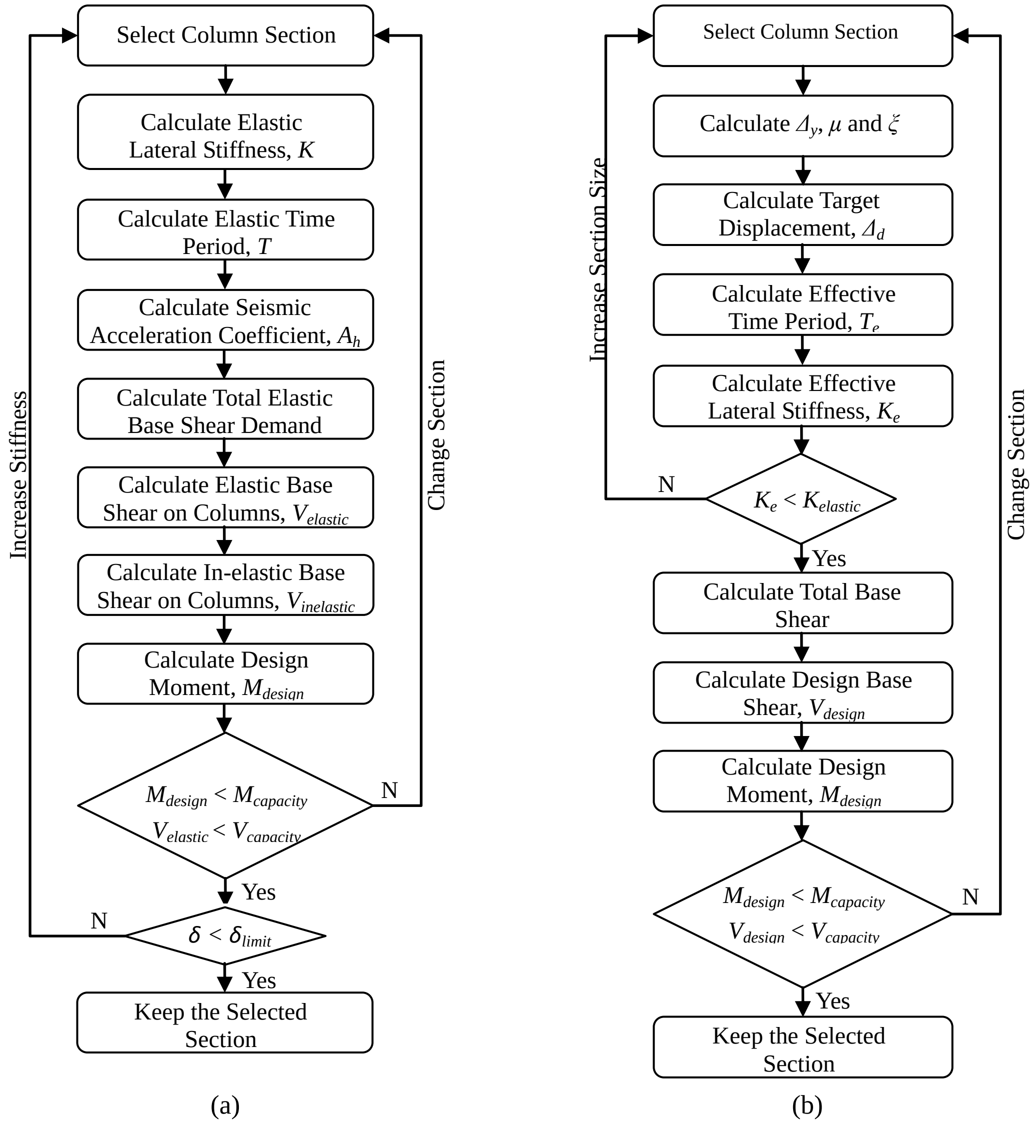


Fig. 4 Flow chart of (a) FBD and (b) DDBD Methodology

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, emphasising the use of displacements rather than forces as a measure of earthquake demand and damage in the structures. Extensive work on the application of capacity design principles to assure ductile mechanisms and concentration of damage in specified regions had also been conducted. Several DBD methodologies have been proposed, among them, the Direct Displacement-Based Design Method has proven to be effective for performance-based seismic design of bridges, buildings and other types of structures (Kowalsky 2002, Ortiz 2006, Suarez and Kowalsky 2006). Fig. 4 shows the design procedure for both FBD and DDBD.

3.1 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 characterisation 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 (Priestley *et al*. 2007). 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 (μ_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 (Priestley *et al*. 2007)

$$\mu_d = 1 + 3\frac{\varphi_p L_p}{\varphi_y H} \tag{1}$$

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.

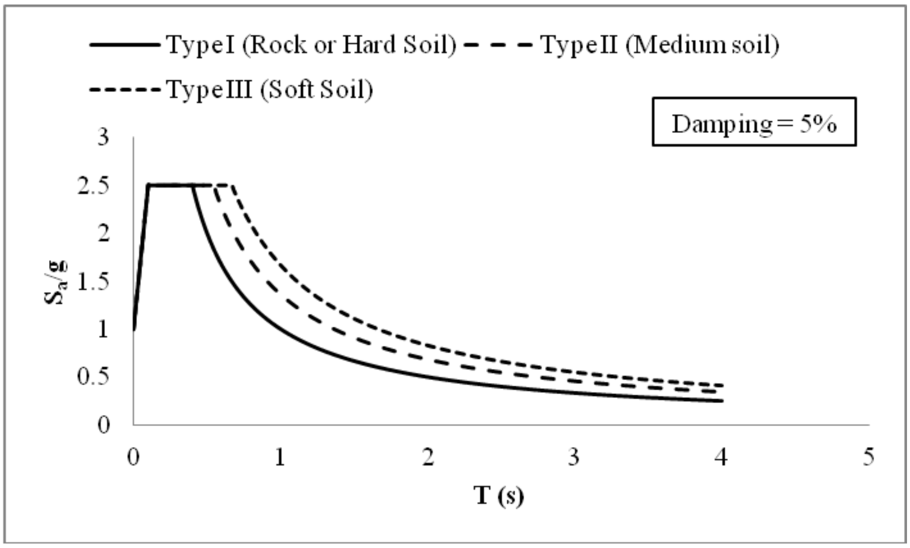


Fig. 5 Response spectrum as per IRC 6 (2010)

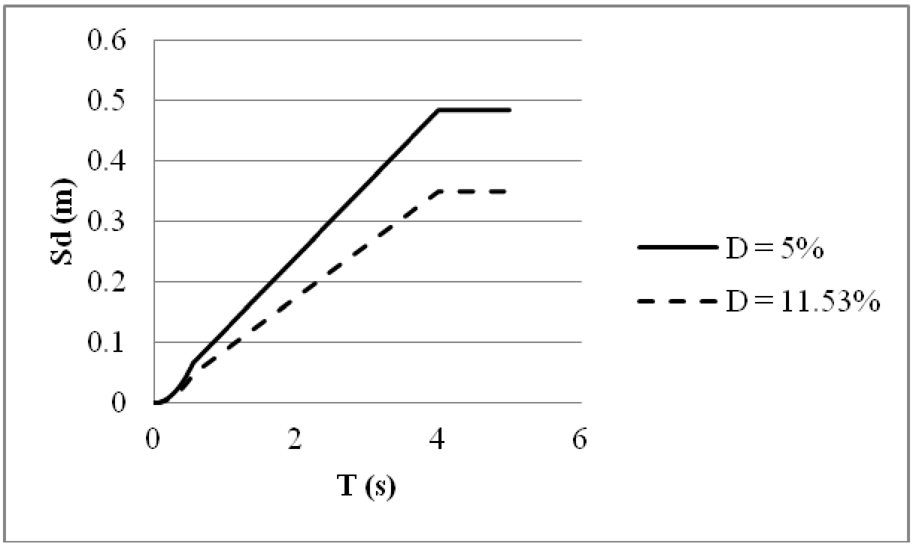


Fig. 6 Displacement response spectra

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 their superstructure design.

3.2 Design of bridges based on FBD and DDBD

The bridge models considered for the comparison of DDBD and FBD methodologies include

Table 1 Design parameters for FBD

RH	T (s)	K_i (kN/m)	V_d (kN)	$V_{inelastic}$ (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
0.5	0.406	409066.7	15078.2	5026.1

Table 2 Design parameters for DDBD

RH	Δ_y	Δ_d	μ_1	μ_2	ξ_1	ξ_2	ξ_{sys}	R_ξ	T_e (s)	W_e (kN)	K_e (kN/m)	V_d (kN)
1	146.3	240	1.64	1.64	0.105	0.105	0.105	0.748	1.72	23238	31610.6	7586.5
0.8	93.63	192	2.05	1.31	0.122	0.084	0.105	0.748	1.72	22878	31120.9	5975.2
0.65	61.81	156	2.52	1.07	0.135	0.059	0.105	0.748	1.72	22608	30753.6	4797.6
0.5	36.57	120	3.28	0.82	0.148	0.05	0.115	0.719	1.38	22338	47203.8	5664.5

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. The response spectrum used in FBD and the displacement response spectra used in DDBD are portrayed in Fig. 5 and Fig. 6, respectively. The bridge is assumed to be situated in Zone V with Type II soil and having an Importance factor (*I*) of 1, corresponding to 'Normal' type of bridges as per IRC 6 (2010). The design parameters used in FBD and DDBD are tabulated in Table 1 and Table 2, respectively in Fig. 7 the base shear at B-1 is found to be increasing as the irregularity in bridges increases, while at B-2 it 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.

In the case of moment variation at bents (refer Fig. 8), 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

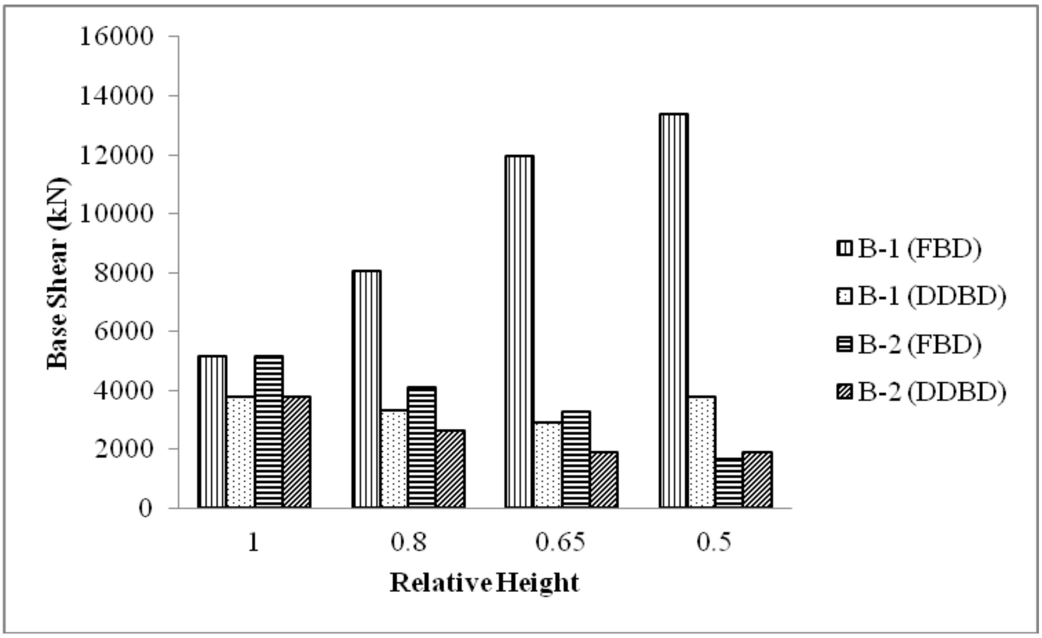


Fig. 7 Variation of base shear at bents with design methodology

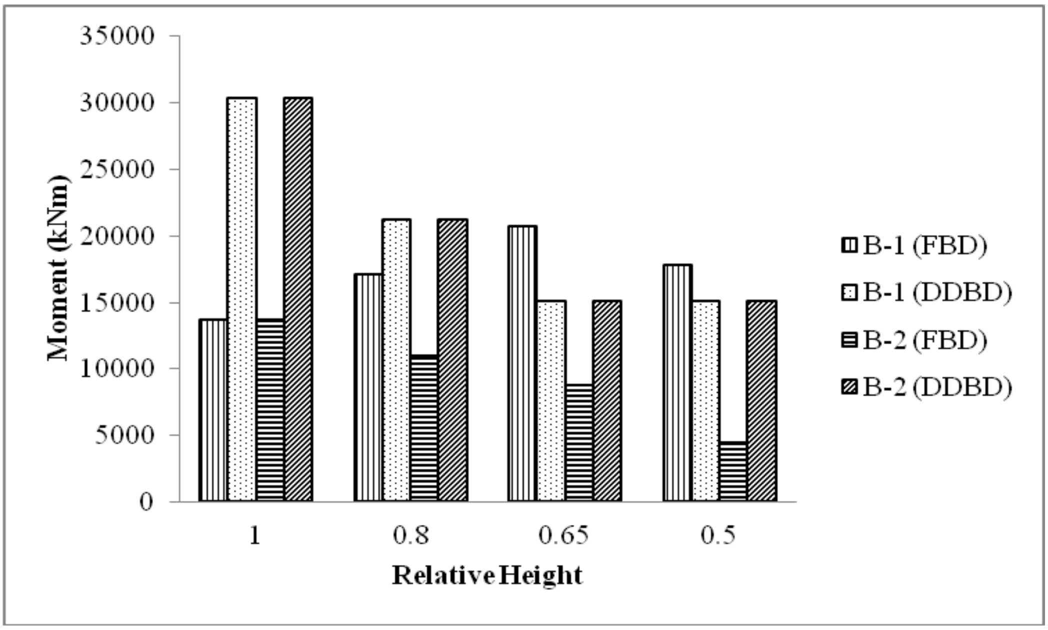


Fig. 8 Variation of moment at bents with design methodology

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.

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

RH	Design Method	Bent-1		Bent-2	
		Longitudinal Reinforcement	**Transverse Reinforcement**	**Longitudinal Reinforcement**	**Transverse Reinforcement**
1	FBD	52-25 mm φ	12 mm φ @ 172 mm c/c	52-25 mm φ	12 mm φ @ 172 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 @ 196 mm c/c
	DDBD	78-25 mm φ	12 mm φ @ 200 mm c/c	78-25 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 @ 231 mm c/c
	DDBD	58-25 mm φ	12 mm φ @ 200 mm c/c	58-25 mm φ	12 mm φ @ 200 mm c/c
0.5	FBD	67-25 mm φ	12 mm φ @ 59 mm c/c	28-20 mm φ	12 mm φ @ 673 mm c/c
	DDBD	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 Reinforcement	**Transverse Reinforcement**	**Longitudinal Reinforcement**	**Transverse 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 Reinforcement	**Transverse Reinforcement**	**Longitudinal Reinforcement**	**Transverse 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#

Tables 3, 4 and 5 shows the reinforcement required as per the three cases discussed in section 3.2. The minimum longitudinal reinforcement percentage specified for FBD as per IRC 112 (i.e., 0.2%) is very small compared to CalTrans (i.e., 1%). For a column section size of 1.5 m×3 m, the minimum reinforcement required as per IRC 112 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). Also previous studies further proved that, column with longitudinal rebar percentage less than 1% has not exhibited sufficient ductility (AASHTO 2012). The minimum confinement reinforcement specified by IRC 112 is found to be higher than CalTrans, which is better from ductility point of view.

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

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

Table 7 Performance levels of bridges designed as per DDBD and FBD

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 8 Performance levels of bridges with revised minimum P_t for FBD

RH	**Design Method**	**Performance Levels**			
		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

Tables 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. Also, the seismic regularity criterion specified by CalTrans (i.e., balanced stiffness approach) can be achieved in bridges designed as per IRC 112 by increasing the minimum column P_t to 1%.

3.3 Comments on Column P_t as per IRC 112

The minimum percentage of column longitudinal reinforcement specified by IRC 112 is found to be very less compared to AASHTO LRFD. AASHTO takes into consideration the seismic regularity concept for irregular columns but, it is left unmentioned in IRC. As the column moment distribution in FBD methodology is inversely proportional to the square of the column height, the longer columns need to resist only 20% of the total column base moment for an RH ratio of 0.5. This leads to significantly less amount of column longitudinal reinforcement in longer columns as shown in Table 3. But, as these columns are designed based on the codal provisions, the strength aspect is satisfied automatically. The commentaries published on International codes such as AASHTO and ACI states that, the minimum longitudinal reinforcement requirement is dependent on the loading, creep and shrinkage aspects also. But, the IRC remain silent on whether the creep and shrinkage aspects are satisfied or not. Based on the numerical and experimental tests conducted by Ziehl *et al.* (1998) for varied grade of concrete and X values (X is the live load to dead load ratio) it is found that, for lower bound value of creep coefficients, the minimum P_t can be brought down from 1% while at the higher bounds, the P_t required is still more than 1%. So, if the worst case is assumed for these factors, the minimum amount of steel that is needed in nearly all the cases to preclude passive yielding of longitudinal reinforcement is more than 1% of the gross cross section (Zhu *et al.* 2007)

Hence, based on the seismic performance evaluation conducted and literatures reviewed, the present study recommends a modification in the minimum column P_t of IRC 112 from 0.2% to 1%, considering irregularity in column heights (Guirguis and Mehanny 2012), loading ratio (live load to dead load ratio), creep and shrinkage effects.

Table 9 Comparison of seismic detailing provisions

Description	IRC 112 (2011)	IRC 21 (2000)	AASHTO (2012)	CalTrans (2013)
Min. P_t	0.2%	0.3%	1%	1%
Max. P_t	4%	8%	4%	4%
Min. φ_L	12 mm	NM	0.625 in.	d_{bl}
Min. φ_T	8 mm	8 mm	NM	NM
S_L	200 mm	NM	8 in.	8 in.
Min. ρ_w (PL)	0.592%	NM	1.16%	To satisfy V_S
Max. S (PL)	125 mm	NM	4 in.	150
Min. ρ_w	0.296%	NM	To satisfy V_S	$\geq$ 50% ρ_w (PL)
Max. S	200 mm	300 mm	12 in.	NM
L_p	1.5 m	NM	3 m	2.25 m

4. Codal comparison

Four codes are taken for the comparison of seismic detailing provisions of bridge columns. Both IRC 112 and IRC 21 are Indian Standards following FBD methodology. While AASHTO and CalTrans are American Standards following FBD and DBD methodology, respectively.

From Table 9 it is clear that the IRC 112 seismic detailing provisions are way better than the previously used IRC 21 (Bhowmick 2014) in terms of Min. P_t, Max. P_t, Min. φ_L, S_L, Min. ρ_w and Max. S. In IRC 21 the need for special zones of ductility or plastic hinges (which is highly desirable for better seismic performance of structures) was completely ignored. Comparing IRC 112 with the other two International Standards (i.e., AASHTO and CalTrans), the main discrepancy is regarding the minimum column P_t recommended by the Indian Standard, which is only 0.2% compared to 1% in the other two codes. Another area of concern is the minimum confinement reinforcement percentage, its spacing and plastic hinge length stipulated by IRC 112 in comparison to AASHTO. This is particularly important as the column axial load capacity after spalling of the concrete cover depends on these three parameters (Boys, Bull and Pampanin 2008, Johnson, Ranf and Saiidi 2008, Papanikolaou and Kappos 2009). The confinement reinforcement is also important in preventing the buckling of longitudinal reinforcement and the plastic hinge length provided will damps out the energy through plastic rotation which triggers redistribution of bending moments. Hence, the IRC 112 need to re-evaluate its stipulation on minimum column P_t, ρ_w, S and L_p considering better seismic performance of the bridges designed using it.

Note: Following are the structural details used in developing Table 9; grade of concrete=M40, grade of steel=Fe 415, diameter of longitudinal rebars=25 mm, column cross-section=1.5 m×3 m and column height=8 m.

Abbreviations: P_t=percentage longitudinal reinforcement, φ_L=diameter of longitudinal bars, φ_T=diameter of transverse bars, S_L=spacing of longitudinal bars along column periphery, ρ_w (PL)=percentage confinement reinforcement at plastic hinge length, S (PL)=spacing of confinement reinforcement at plastic hinge length, ρ_w=percentage confinement reinforcement at regions other than plastic hinge length, S=spacing of confinement reinforcement at regions other than plastic hinge length, L_p=plastic hinge length, NM=not mentioned, d_{bl}=nominal diameter of longitudinal reinforcement, V_S=nominal shear strength.

5. 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 targeted performance objectives. 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. The study also compares the seismic detailing provisions of IRC 112, IRC 21, AASHTO LRFD and CalTrans and recommends modification in the minimum percentage longitudinal reinforcement of bridge columns based on seismic performance evaluation and literature review.

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. A comparative study on FBD and DDBD methodologies reveal that it is better to adopt the latter particularly for bridges with irregular column height. It is also found that, even though the seismic detailing provisions for confinement reinforcement of IRC 112 is better than the previously published IRC 21, the minimum longitudinal reinforcement percentage advocated in IRC 112 needs to be re-evaluated considering the possible irregularity in column heights along with the inherent creep and shrinkage effects. The present study recommends an increase in minimum column P_t to 1% of gross cross-sectional area from 0.2% in IRC 112 considering better seismic performance.

References

AASHTO LRFD (2012), *American Association of State and Highway Transportation Officials LRFD Bridge Design Specifications*, Washington DC.

Akbari, R. (2010), "Cyclic response of RC continuous span bridges with irregular configuration in longitudinal direction", *Structure and Infrastructure Engineering*.

Bhowmick, A. (2014), "Detailing provisions of IRC:112-2011 compared with Previous codes (i.e., IRC:21 & IRC:18)", *J. Indian Roads Cong.*, **75**(1), 610.

Boys, A., Bull, D. and Pampanin, S. (2008), "Seismic performance assessment of inadequately detailed reinforced concrete columns", *New Zealand Society of Earthquake Engineering (NZSEE) Conference*, Wairakei, New Zealand.

Guirguis, J.E.B. and Mehanny, S.S.F. (2012), "Evaluating codes criteria for regular seismic behavior of continuous concrete box girder bridges with unequal height piers", *J. Bridge Eng.*, **18**(6), 486-498.

Ishac, M.G. and Mehanny, S.S.F. (2016), "Do mixed pier-to-deck connections alleviate irregularity of seismic response of bridges with unequal height piers?", *Bull. Earthq. Eng.*, 1-25.

Johnson, N., Ranf, R.T., Saiidi, M.S., Sanders, D. and Eberhard, M. (2008), "Seismic testing of a two-span reinforced concrete bridge", *J. Bridge Eng.*, **13**(2), 173-182.

Kowalsky, M.J. (2002), "A displacement based approach for the seismic design of continuous concrete bridges", *Earthq. Eng. Struct. D.*, **31**(3), 719-747.
Ortiz, J. (2006), "Displacement-based design of continuous concrete bridges under transverse seismic excitation", *European School for Advanced Studies in Reduction of Seismic Risk (ROSE School).*
Papanikolaou, V.K. and Kappos, A.J. (2009), "Numerical study of confinement effectiveness in solid and hollow reinforced concrete bridge piers: Methodology", *Comput. Struct.*, **87**(21), 1427-1439.
Priestley, M.J.N. (2000), "Performance based seismic design", *Proceedings of the 12th World Conference on Earthquake Engineering*, New Zealand.
Priestley, M.J.N., Calvi, G.M. and Kowalsky, M.J. (2007), *Displacement-based seismic design of structures, IUSS Press*, Pavia, Italy, ISBN:978-88-6198-000-6
Suarez, V. and Kowalsky, M. (2006), "Implementation of displacement based design for highway bridges", *Proceedings of the fifth National Seismic Conference on Bridges & Highways*, San Francisco, Canada.
Tamanani, M., Gian, Y. and Ayoub, A. (2016), "Evaluation of code criteria for bridges with unequal pier heights", *Bull. Earthq. Eng.*, 1-24.
Zhu, L., Elwood, K.J. and Haukaas, T. (2007), "Classification and seismic safety evaluation of existing reinforced concrete columns", *J. Struct. Eng.*, **133**(9), 1316-1330.
Ziehl, P.H., Cloyd, J.E. and Kreger, M.E. (1998), "Evaluation of minimum longitudinal reinforcement requirements for reinforced concrete columns", Report No. FHWA/TX-02/1473-S, Center for Transportation Research The University of Texas at Austin.

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