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Performance-based seismic design and evaluation of steel eccentrically braced frames with tubular links as bridge bents

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ABSTRACT

The latest release of the Canadian Highway Bridge Design Code (CHBDC), S6-14, incorporates performance-based design (PBD) provisions for bridges in Canada for the first time. The focus of this study is on ductile eccentrically braced frames (EBFs) as bridge substructure. For member proportioning, the CHBDC S6-14 refers to the Canadian steel design standard for buildings, CSA S16-14, stating a force reduction factor, R=4. This is force-based design (FBD), and there is need to evaluate the design in terms of the performance descriptions and damage states by carrying out the analyses recommended by CHBDC S6-14. For this case study, an existing bridge is considered as a Major Route bridge, and an EBF with built-up tubular shear link has been chosen as an earthquake-resisting system (ERS). Four different cases have been designed including two using FBD and two for PBD approach for comparison purposes. Due to the lack of strain/rotation criteria in CHBDC S6-14 for EBFs as bridge bents, different acceptance criteria for rotations and corresponding damage states along with different methods of repairs have been proposed from the literature review. The response spectrum analysis coupled with inelastic static pushover analysis is used for global displacement demands and for demonstrating local component performance compliance of shear links. Nonlinear time-history analysis is also used to check and provide a comparison of the first approach. The code requires no-yielding for the 475-year return period event. This criterion governs the design and makes the sizes large and inefficient, while the link plastic rotations corresponding to higher return period events are very low compared to the allowable limits provided in the literature for links mainly used in buildings. Through different cases, it is demonstrated that if the links are made replaceable and allowed to have limited yielding at 475-year earthquake, it makes the design more practical.

Keywords: Performance-based Design, Canadian Highway Bridge Design Code, Steel Bridges, Eccentrically Braced Frames, Tubular Shear Links.

INTRODUCTION

The latest release of the Canadian Highway Bridge Design Code (CHBDC), S6-14, incorporates performance-based design (PBD) provisions for bridges in Canada for the first time. Until recently, the main design goal has been life safety with designs mostly based on strength criteria which is the main concept of force-based design (FBD) approach. There has been a gradual shift from 'strength-based design' to 'performance-based design' and a recognition that strength is not always equal to better performance. Moreover, the increase in strength does not essentially mean higher safety, nor does it imply less damage [1].The focus of this study is bridges with steel substructure such as ductile eccentrically braced frames (EBFs). For member proportioning, CHBDC (S6-14) refers to the Canadian steel building code S16-14, using a force reduction factor, R=4. This design approach is force-based and post-earthquake performance of the bridge cannot be quantified using this approach. There is a need to assess the design in terms of performance descriptions and damage states by carrying out the analyses recommended by CHBDC S6-14. Moreover, CHBDC S6-14 does not provide clear guidelines to check whether such performance objectives are achieved, and there is limited literature available on this issue so far, especially for bridges with ductile steel sub-structures.

PBD Criteria and Analysis Requirements by CHBDC S6-14

For each of three bridge importance categories (Lifeline bridges, Major Route bridges and Other bridges), CHBDC S6-14 specifies service and damage levels required to be fulfilled for multiple hazard levels including 475-, 975-, and 2475-year return period events. For each hazard level, CHBDC S6-14 specifies required analyses corresponding to each seismic performance category (SPC) and importance category of bridge. For Major route bridges in SPC of 3, the required analysis by CHBDC S6-14 for 475-years return-period event is elastic dynamic analysis (EDA). For 975-year and 2475-year return period events, CHBDC S6-14 requires EDA as well as inelastic static push-over analysis (ISPA). For this study, in addition to analyses required by CHBDC S6-14, a complementary nonlinear time-history analysis (NLTHA) will be used to check and provide a comparison of EDA and ISPA. The performance criteria for multiple performance levels and damage conditions are provided in CHBDC S6-14 [2]. As the focus of this study is mainly steel substructure bridges, specified performance criteria for these bridges are given Table 1.

Table 1 Performance Criteria for Steel Bridges as per CHBDC S6-14 (CSA Group, 2014a)

Level	Service	Damage	Criteria
1	Immediate	Minimal damage	- Essentially elastic with minor damage - Steel strains (ε_{st}) ≤ yield strain (ε_y) - No local or global buckling
2	Limited	Repairable damage	- Full dead plus live load-carrying capability - No buckling of primary members - Secondary members may buckle without causing instability
3	Service Disruption	Extensive damage	- Full dead plus 50% live load-carrying capability - No global buckling of gravity-load-supporting elements

Eccentrically Braced Frames as Earthquake-Resisting System (ERS)

EBFs are lateral-load-resisting systems whose primary purpose is to dissipate energy in the event of an earthquake through yielding of a small segment called a link element, usually between the ends of two braces as shown in Figure 1. In oreder to avoid the out-of-plane buckling of the link member, a new form of tubular link section made of built-up steel plates was tested and validated; it was found that tubular sections do not need lateral bracing against lateral torsional buckling [3]. EBF towers made of built-up tubular links were implemented as temporary towers in San Francisco Oakland Bay Bridge [1]. As only the link beam is expected to deform inelastically in an EBF, while all other frame members are intended to remain within elastic limits. Due to this reason, the performance criteria given in CHBDC S6-14 seem inapplicable for EBFs as bridge bents. There is a need to specifically define the damage states for nonlinear behaviour of link elements. For instance, the criteria underlined in Table 1 refer to load-carrying capacity of a substructure. In fact, all the capacity-protected members resisting gravity loading will probably remain in the elastic range and would not buckle until the P-delta effect caused huge drifts in the structure.

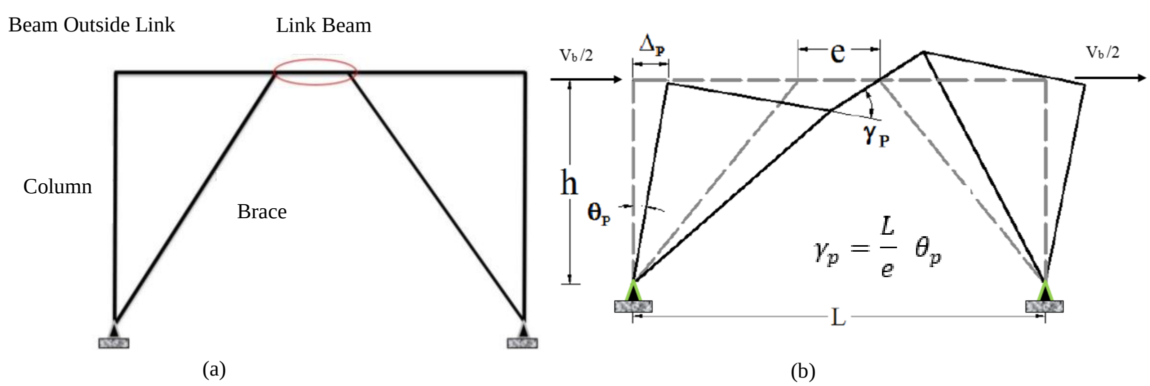

Figure 1 Typical EBF Geometric Configuration: (a) Chevron EBF shape, (b) deformed shape, Adapted from: [3].

PROPOSED ACCEPTANCE CRITERIA FOR EBFS AS BRIDGE PIERS

There is a need to carry out performance evaluation of EBFs mainly based on repair cost and business interruption, by relating the damage limits with demand parameters such as link plastic rotation in an EBF [4]. Damage states can be directly related to the failure mode of a steel member [4]. Based on the work done by Gulec et al. (2011) related to damage states and fragility functions for shear link beams in EBFs, shear link total rotations for a variety of damage states are reviewed. The researchers used the plastic link rotation as a demand parameter for damage evaluation. They correlated the experimental test results with different damage states and recommended different suitable methods of repair. For a consistent approach, total link rotation is used for acceptance criteria limits instead of plastic rotation. The total link rotation can be calculated as: $\gamma_T = \gamma_E + \gamma_P$. Here, γ_T is the total link rotation, i.e., the ratio of total relative vertical displacement at the end of a link to the length (e) of the link member. γ_E is the elastic link rotation that can be calculated using theoretical equations provided in FEMA 356 [5]. γ_p is the link plastic rotation, which is the inelastic component of the rotation of the link member relative to the beam outside the link as shown in Figure 1. Here, L is the total frame bay width, e is the link member length and θ_p is the storey plastic rotation angle The link plastic rotation can be calculated from the rigid plastic mechanism. As all other framing members are designed to remain elastic, deformations from beam outside the link are not considered by assuming that it will stay principally elastic while the link is subjected to large plastic deformations [3]. The proposed acceptance criteria limits with total link rotation as a demand parameter obtained from literature are provided in Table 2 and will be used for performance evaluation of different EBF bridge bents for this study. Once the damage states have been defined and related to the demand parameters, different repair types are linked with visible damage type to estimate the serviceability of a bridge after an earthquake.

Table 2 Proposed acceptance criteria and method of repairs (MOR) for each damage state (Gulec et al., 2011)

Level	Service	Damage	Shear Link Total Rotation (radians)	Damage States	Method of Repair	Repair Action
1	Immediate	Minimal damage	0.015	- Web yielding - Flange yielding - Stiffener yielding	Cosmetic repair MOR-0	No structural repair required; repaint structural steel $\gamma_T \leq 0.015$ radians
2	Limited	Repairable damage	0.06	- Web local buckling - Flange local buckling	Heat straightening MOR-2	Provide heat straightening in the immediate area of web and flange local buckling $\gamma_T \leq 0.06$ radians
3	Service Disruption	Extensive damage	0.08	- Web fracture - Flange fracture - Lateral torsional buckling	Link replacement MOR-3	Replace link by flame cutting and weld new link section $\gamma_T \leq 0.08$ radians

CASE STUDY

The two-span Sombrio Bridge located on Vancouver Island, British Columbia, with a total span of 122 m is selected as a case study, and an EBF has been chosen as the substructure to replace a two-column concrete bent as shown in Figure 2. This existing bridge consists of two unequal spans of 40 m and 82 m. For this case study, the bridge is considered as a regular Major Route bridge. Based on existing drawings, site class C has been considered. It is confirmed that the contribution of bent to longitudinal restraint is minor, and abutments are mainly contributing for longitudinal restraint. The substructure EBF bent is therefore designed for seismic loads in the transverse direction only. The tributary seismic mass of the existing superstructure is applied to a single bent. The structure is therefore modelled as a single-degree-of-freedom (SDOF) system as it has been confirmed from practical projects that this approach gives a reasonable estimate of the dynamic behaviour of a bridge when compared with the detailed model including the superstructure [6]. The steel material selected for ductile EBFs is CSA G40.21, Grade 350W, with specified minimum yield stress Fy of 350MPa as permitted CHBDC S6-14 [2].

For this study, the two exterior girders are placed directly above the columns, while the two interior girders are placed at the ends of the shear link, as shown in *Figure 3*. This geometry helps transfer dead load (SLS for each girder = 3470 kN) to foundations using columns and braces without putting high demands on cap-beams from gravity loading. Superstructure bridge geometry (location of girders) affects the member sizing. High gravity loading on a beam outside the link will require a more significant section. The Canadian steel design standard CSA S16-14 does not give any recommendations regarding replaceable links made of built-up tubular sections. Due to the requirement of CSA S16-14 for continuous link beam for the built-up tubular section, i.e., same link section as beam outside, it might require a more significant link section, causing a considerable increase in all other capacity-protected members designed for forces generated by fully yielded and strain-hardened link. Therefore, it is decided to consider one design case with replaceable link beam for this study. The center to center (c/c) distance between girders is 3 m. This dimension is a constraint due to the geometry of the existing structure. Due to this restriction, shear link length is taken as 3 m for the case where the link section is the same as the beam outside (continuous link), and 2.4 m for a case where the link is made smaller than the outside beam portion (replaceable link). There is a lack of literature for studies related to the design of EBFs for bridge piers according to Canadian design provisions [2], [7], [8].According to CHBDC S6-14, FBD is required for regular Major Route bridges with seismic performance category of 3 with a condition that, PBD might be required by the Regulatory Authority for this case. Another clause states that PBD may be used for all cases. Therefore, the bent will be designed using both design approaches, i.e. PBD and FBD for comparison purpose.

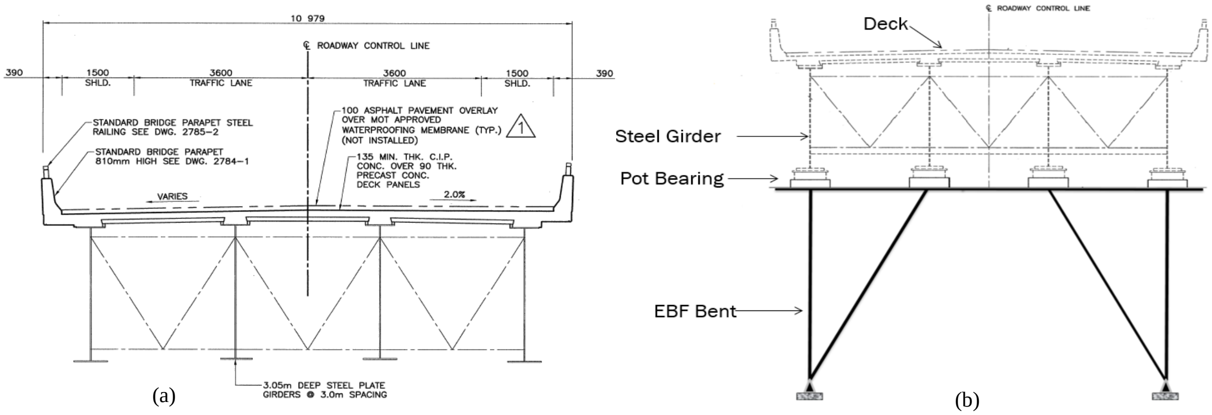

Figure 2 Sombrio Bridge: (a) superstructure typical section [9] *(b)replaced EBF bent .*

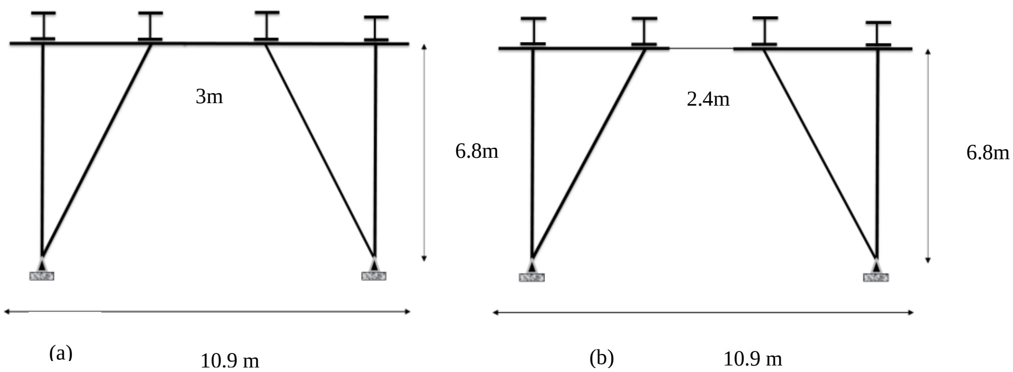

Figure 3 Selected EBF geometry with girders orientation: (a) for design case 1, 2 and 3, (b) for design case 4.

Performance-Based Design Approach

The PBD methodology used for two cases is given in Figure 4 For the selected case study site location with geographical coordinates of 48.4952 ºN and 124.2584 ºW, uniform hazard spectrum (UHS) values corresponding to 475-, 975-, and 2475-year return period event are provided in Figure 4.

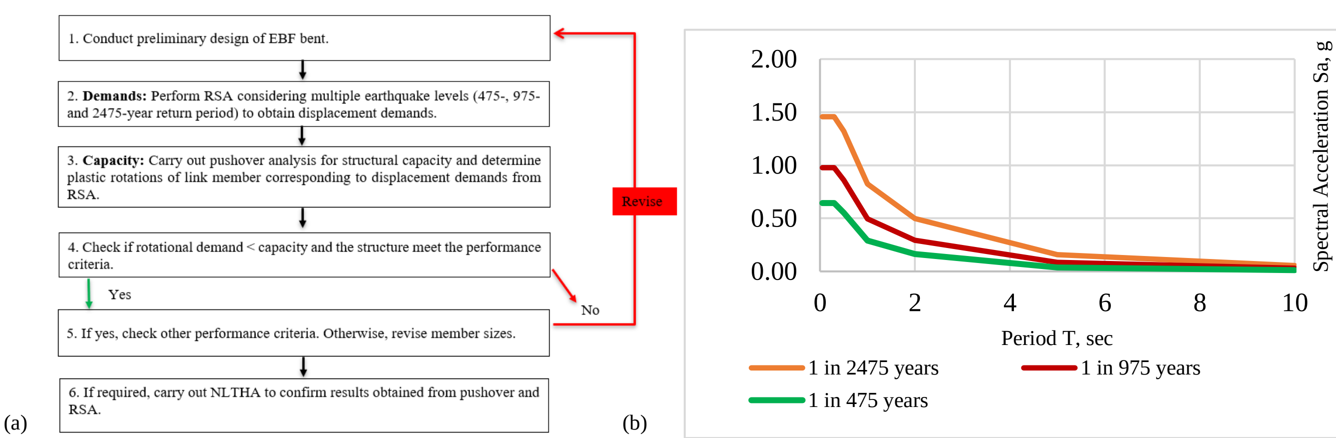

Figure 4 PBD of EBF Bents: (a) design methodology (b)5% damped Sombrio UHS

EBF Bent Design

Once the seismic design forces have been determined by both FBD and PBD approaches, the next step is to size the link member in an EBF which will behave as a structural fuse. The rest of the members are designed afterwards to stay elastic for the forces applied by the link in its entirely yielded and strain-hardened state. The link length plays a crucial role for all member sizes as well as the inelastic response of an EBF. Corresponding to each type of yielding, CSA S16-14 gives a maximum inelastic rotation limits (γ_p) of 0.08 radians for shear links, 0.02 radians of flexural links and linear interpolation is recommended for intermediate links. Shear links are preferred due to their high energy dissipation capacity as well as ductility.

A total of four different designs of bents were carried out for comparison purposes. All member sizes for designed bents are given in Table 3 along with the corresponding design approach and hazard level. Here d, w, tf, and tw correspond to the overall depth of the section, the overall width of the section, the thickness of flange and thickness of web respectively.

Table 3 Considered EBF design cases

Design Case	D1	D2	D3	D4
Design Approach	FBD(I_E = 1.5)	FBD (I_E = 1.0)	PBD	PBD
Hazed Level (Return period)	2,475	2,475	475,975 and 2,475	475,975 and 2,475
Link Length e (mm)	3000	3000	3000	2400
Link Type	shear	Shear	shear	shear
Link Type	Non-replaceable	Non-replaceable	Non-replaceable	Replaceable
Braces Intersection e' (mm)	3000	3000	3000	3000
Link Size (d x w x t_f x t_w)	750x800x54x20	600x700x50x17	750x900x57x25	600x600x40x15
Beam Size (d x w x t_f x t_w)	750x800x54x20	600x700x50x17	750x900x57x25	750x700x55x25
Brace Size (d x w x t_f x t_w)	650x650x40x40	700x700x40x40	800x800x55x55	600x600x25x25
Column Size (d x w x t_f x t_w)	600x250x30x30	600x250x30x30	700x350x30x30	500x400x25x25

NUMERICAL MODELING

SAP2000 version 20 is used as the platform for numerical modelling and analyses of EBFs. Capacity design is a core requirement for an EBF; therefore, all other members except shear links are modelled as elastic elements as they are not expected to experience plastic deformation. Girders have been arranged such that gravity loads from superstructure are applied on beams outside the link member. To validate the numerical modelling approach for prediction of nonlinear link behaviour, a calibration procedure is carried out. Berman and Bruneau (2007) reported the results of a proof-of-concept test setup consisting of full-scale single-panel EBF bent with a built-up hybrid tubular cross-section [3]. The term hybrid means that a link cross-section has different web and flange yield strengths. The full-scale single panel EBF bent is modelled in SAP 2000 with the same geometry, and member sizes and same loading protocol is applied on the bent according to Berman and Bruneau (2007).

The approach considered is to model plastic hinges in the link member to capture nonlinear behaviour by using the nonlinear modelling parameters as recommended by ASCE41-13 [10]. A deformation-controlled shear (V2) hinge with force-displacement type is assigned in the middle of the link member where shear is expected to be maximum. A default kinematics hysteresis model is considered for this case that does not require any additional parameters input in SAP2000 [11]. One drawback of using the plastic hinge method is that while defining the hysteresis model in SAP2000, the user does not have the option to change parameters to exactly match the hysteresis behaviour of actual test results. Regardless of that, this method gives a good match to the actual backbone curve from the test as shown in Figure 5.

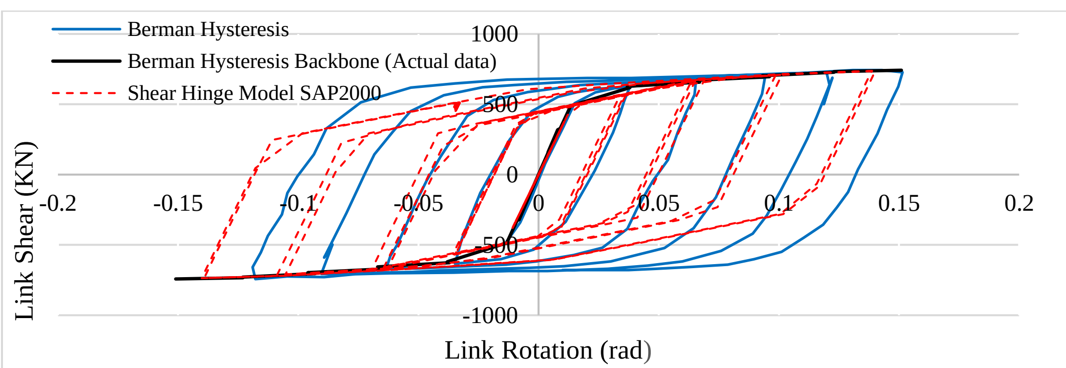

Figure 5 Experimental hysteresis calibration using shear plastic hinge

SEISMIC EVALUATION

Response Spectrum Analysis and Pushover Analysis

A dynamic (response spectrum) analysis (EDA) coupled with inelastic static pushover analysis (ISPA) is carried out for global demands and for demonstrating local component performance compliance of shear link. The modal damping is considered as 5% for RSA. The considered bridge is a regular bridge with the fundamental mode of vibration governing the response and well-separated frequencies. The displacement demands at the top right edge of the bent are monitored for bridge site-specific response spectrum. The fundamental periods of all designed bents are 0.46 sec, 0.53 sec, 0.39 sec and 0.57sec for D1, D2, D3, and D4 respectively. For these design cases with a low fundamental period, the structure lies in the acceleration- sensitive zone of the spectrum. The equal displacement principle is of doubtful validity in this zone; therefore, modified displacement demands from FEMA 440 [12] displacement modification method is used for further performance evaluation of EBF bents. For ISPA, ASCE 41-13 nonlinear modeling parameters are used to define the plastic shear hinge in the middle of the shear link. The RSA displacement demands from 475-, 975-, and 2475-year return period response spectrum analyses are superimposed on pushover capacity curve after applying the displacement correction to evaluate the performance of bents at required performance levels. Link plastic rotations have been calculated corresponding to RSA displacement demands and presented as total link rotations by adding the elastic rotation component. Figure 6 shows the pushover curves with displacement demands from 475-, 975-, and 2475-year return period response spectrum analyses. On the pushover curves, acceptance criteria limits for damage states concerning minimal damage, repairable damage, and extensive damage have also been provided.

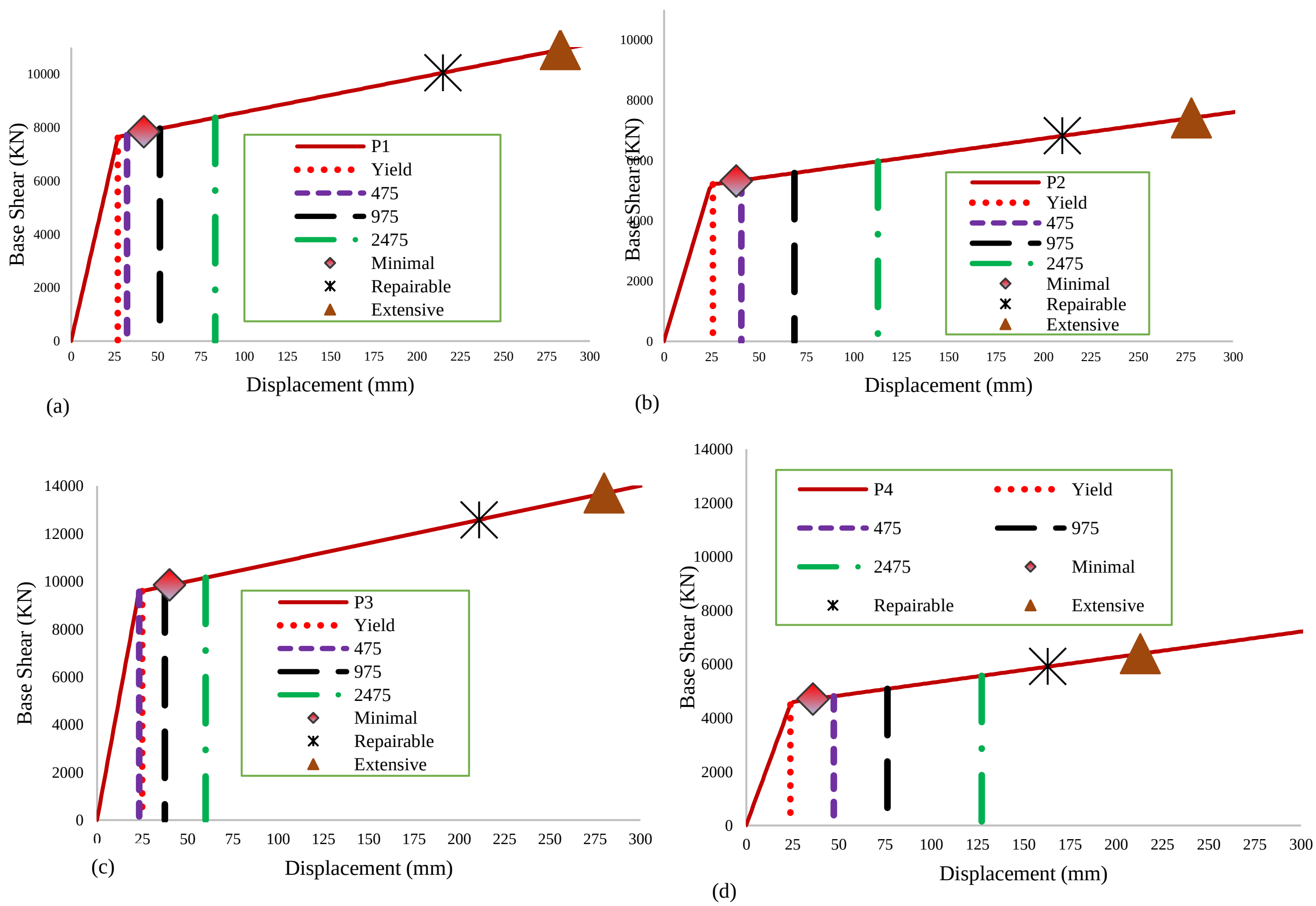

Figure 6 Pushover curves RSA demands and acceptance criteria limits: (a) Design 1, (b) Design 2, (c) Design 3, (d) Design 4

Nonlinear Time-history Analysis (NLTHA)

The nonlinear dynamic analysis is carried out by using the spectrally matched time-histories to evaluate the performance of four EBF bents at different hazard levels and to compare the results with pushover and response spectrum analyses. Like all other analyses, serviceability limit state (SLS) loads from the superstructure are applied at each girder location to capture the seismic mass in SAP2000 models. The damping is assigned as 2% Rayleigh damping in the first two modes. A plastic shear hinge is assigned in the middle of the link member to capture the nonlinear behaviour of the yielded link member. All other members are capacity-protected and modelled as elastic elements. For comparison purposes, bent global modified displacement demands from RSA, link total rotations (θE + θp) obtained from pushover capacity curve corresponding to the RSA demands as well as link total rotations from NLTHA are provided in Table 4. The total link rotation is provided as an average of 11 ground motion responses at each hazard level. To evaluate these results, proposed acceptance criteria limits for minimal, repairable, and extensive damage have also been provided.

Table 4 Link total rotations from RSA/Pushover and NLTHA

Design Case	**Service**	**Load Case**	**RSA Disp. (mm)**	**Total Link Rotation Pushover/ RSA (radians)**	**Total Link Rotation NLTHA (radians)**	**CHBDC S6-14 Acceptance (Yes or No)**	**Acceptance Criteria Rotation (radians)**	**Proposed Acceptance (Yes or No)**	**Method of repair**
D1	Immediate	1.25D+EQ_475	32	0.007	0.009	No	0.015	Yes	MOR-1: cosmetic
D1	Limited	1.25D+EQ_975	51	0.013	0.014	Yes	0.060	Yes	MOR-2; heat straightening
D1	Disruption	1.25D+EQ_2475	83	0.022	0.022	Yes	0.080	Yes	MOR-2; heat straightening
D2	Immediate	1.25D+EQ_475	41	0.010	0.011	No	0.015	Yes	MOR-1: cosmetic
D2	Limited	1.25D+EQ_975	68	0.018	0.015	Yes	0.060	Yes	MOR-2; heat straightening
D2	Disruption	1.25D+EQ_2475	112	0.031	0.025	Yes	0.080	Yes	MOR-2; heat straightening
D3	Immediate	1.25D+EQ_475	23	0.005	0.006	Yes	0.015	Yes	No repair required
D3	Limited	1.25D+EQ_975	37	0.009	0.010	Yes	0.060	Yes	MOR-1; cosmetic
D3	Disruption	1.25D+EQ_2475	60	0.016	0.017	Yes	0.080	Yes	MOR-2; heat straightening
D4	Immediate	1.25D+EQ_475	47	0.014	0.015	No	0.015	Yes	MOR-1: cosmetic
D4	Limited	1.25D+EQ_975	76	0.026	0.025	Yes	0.060	Yes	MOR-2; heat straightening
D4	Disruption	1.25D+EQ_2475	127	0.046	0.046	Yes	0.080	Yes	MOR-2; heat straightening

Performance Evaluation from Pushover-RSA and NLTHA Results

The results from NLTHA analysis have been compared with total link rotations obtained from pushover analysis corresponding to modified RSA displacement demands and found to be in close agreement. By evaluating the total link rotations from pushover/RSA and NLTHA and comparing them with CHBDC S6-14 acceptance criteria and proposed acceptance criteria, some observations are: D1 and D2 do not meet the no-yielding criterion by CHBDC S6-14 corresponding to the 475-year return period event. Although the seismic demands increase for the 975-year and 2475-year return period events, the increase is such that the 475-year hazard and the corresponding performance criteria govern the design. D1 and D2 fulfil the proposed acceptable limit for immediate service. By comparing this with the damage states and methods of repair proposed, MOR-0 cosmetic repair is required for this damage state. Design 3 design fulfills the no-yielding criterion of CHBDC S6-14. It also fulfils the proposed acceptable limit for immediate service without having yielding at 475-year return period event. No repair work is anticipated for this design at 475-year return period as the link is fully elastic at this hazard level. Design 4, which is also based on satisfying PBD performance criteria with a replaceable link, meets the proposed acceptance criteria limit for minimal damage and extensive damage for higher return period events. This design does not meet the no-yielding criterion by CHBDC S6-014. MOR-2 heat straightening will be required for this damage state. D4 demonstrates that if the links are made replaceable and allowed to have limited yielding at 475-year return period, the design is more practical.

According to CHBDC S6-14, the designer has the option to adopt either FBD approach using an importance factor (I_E) of 1.5 or PBD approach for Regular Major Route Bridges. The performance of structures designed using the FBD approach is expected to be consistent with PBD at the 2475-year return period as per CHBDC S6-14. The code does not require any design checking at lower hazard levels (475-,975-year return period) for structures designed using FBD approach. It has been observed that the member sizes for D3 (PBD) are quite different than D1 (FBD). Moreover, both FBD cases (D1 and D2) and the PBD case (D4) do not fulfil minimal damage performance criteria by CHDBC S6-14. Table 4 show the evaluation of performance criteria based on proposed acceptance criteria as well as CHBDC S6-14 acceptance criteria. A summary of different methods of repair (MOR) has been provided corresponding to each damage state.

CONCLUSIONS

The 475-year "no-yielding criterion" governs the design, with very low link plastic rotations corresponding to higher return-period events. If the links are made replaceable and allowed to have limited yielding at 475-year hazard, it makes the design more practical. Therefore, there is a need to re-calibrate the 475-year return period related no-yielding criteria in S6-14 for obtaining practical designs. More guidance needs to be provided for rotational limits for higher return-period events corresponding to more significant member sized that would usually be required for EBFs supporting bridge superstructures in comparison to buildings. Moreover, CHBDC S6-14 does not provide any information regarding explicit performance check at lower hazard levels (475 & 975 years return periods) using FBD approach.

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