



Seismic Risk Assessment of Buckling Restrained Braced Frame Buildings, Including Detailed Evaluation of Residual Drifts

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ABSTRACT

This paper discusses a method for the efficient yet detailed seismic risk assessment of buckling restrained braced frame (BRBF) buildings and then shows how this approach can be used for resilience design of BRBF buildings. This assessment process uses the FEMA P-58 risk assessment framework and includes a method to predict seismic structural responses without needing to build a full detailed nonlinear structural model. For structural responses, residual drifts are investigated in detail and the study shows the beneficial effects that a moment resisting back-up frame, as well as gravity system components, have on reducing residual drift demands in BRBF systems, and thereby increasing their seismic resilience.

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1 INTRODUCTION

Buckling restrained braced frames (BRBFs) are a special class of concentrically braced frames that can yield in both compression and tension, resulting in a highly ductile structural system with equal strength and stiffness in each direction and without degradation in energy absorption and slackness developing under inelastic cyclic action. The structural response prediction method discussed here covers CoreBrace BRBF buildings from 1-story to 25-stories, without irregularities, and with a range of standard bracing layouts. The scope also includes treatment of the back-up frame and the positive effect of the gravity system. The method covers response prediction for levels of ground motion ranging from elastic response to highly nonlinear response above MCE levels.

2 CREATION OF METHOD TO PREDICT NONLINEAR STRUCTURAL RESPONSES WITHOUT A STRUCTURAL MODEL

2.1 Structural design and model sets considered

The designs used in this assessment were taken from the archetypes used in the NIST GCR 10-917-8 report, Evaluation of the FEMA P-695 Methodology for Quantification of Building Seismic Performance Factors [1]. Structural bracing configurations consisting of single diagonal and two-story X brace configurations were utilized for the development of the method. Table 1 summarizes the structural layouts considered for each of the two framing configurations and example floor plans are shown in Figure 1. Six variants summarized in Table 2 were run for each model to investigate the effect of the backup frame presence, the base connection, and the presence of additional stiffness and restoring force from the gravity system (including both shear tab behavior in the gravity system bays and also gravity column continuity). The resulting analysis set includes a total of 90 building designs and modelling variants.

Each nonlinear building model was analyzed on the OpenSees [2] modeling platform using the FEMA P695 ground motion set (a standardized set of 44 far-field motions). Incremental nonlinear dynamic response-history analysis was then completed from small ground motions (with elastic structural behavior) up to large ground motions (e.g. MCE highly nonlinear response). The structural responses that are tracked in the analysis are interstory drifts, peak floor accelerations and residual drifts. Uncertainties and correlations in the response values were also tracked.

Table 1: Summary of structures used for the development of the structural response method

Single Diagonal Bracing		2 Story X-Bracing	
Stories	Bay Width (m)	Stories	Bay Width (m)
1	7.62	2	6.10
2	4.57	2	9.14
3	7.62	3	9.14
4	4.57	6	9.14
6	4.57	12	6.10
9	4.57	12	9.14
12	4.57		
18	4.57		

Table 2: Variants considered for each model

Variant	Backup Frame	Base Connection	Gravity System
1	None	Pinned	None
2	None	Pinned	Present
3	Present	Pinned	None
4	Present	Pinned	Present
5	Present	Fixed	None
6	Present	Fixed	Present



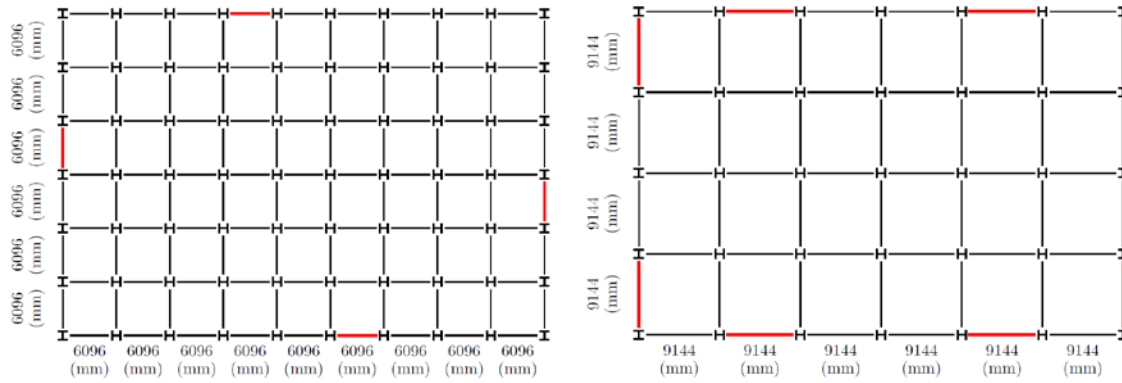


Figure 01: Representative plan views of structures for 2 story (left) and 12 story (right). The 3-9 story archetypes have the same layout as the 12 story shown, but with only two braced frames in each direction rather than four.

2.2 Method for predicting peak interstory drift (IDR)

The roof drift of the structure is determined using ASCE 41 (δ_t from Equation 3-14 of ASCE 41-13) and then modified based on the number of stories in the structure to agree with the results of the nonlinear time history analysis performed for this study. The principal mode shape of the structure is then scaled such that the target roof drift is met. This is the IDR_elastic shown in Figure 2.

When the structure is subject to shaking that causes inelastic response, the inelastic drift tends to localize in the lower levels of the structure. To reflect this in the peak IDR profile, the drift in the lower levels of the structure are increased relative to IDR_elastic (i.e. the damage is localized in these lower stories) and the upper levels are decreased relative to IDR_elastic (i.e. the upper stories have less drift, due to the localization in the lower stories) in such a way that the total roof drift is unchanged. This method for localization of drifts in the bottom stories is calibrated to the large database on nonlinear response-history analysis results and the level of localization is dependent on the ratio of ground shaking demand to the strength of the building (per the S value described below). This results in the IDR_mod shown in Figure 2.

The degree to which the peak IDR is localized is dependent on the strength ratio, S , defined as

$$S = \frac{S_a}{(V_y/W)}$$

Where S_a is the spectral acceleration (in g) at the fundamental period of the building at the assumed hazard level, V_y is the base shear capacity of the building, and W is the total weight of the building.

The location of the “lower” and “upper” levels depends on the presence of a backup frame and, if the backup frame is present, the end condition at the base of the backup frame. The end conditions at the bottom of the backup frame considered were both pinned and fixed columns, to be determined at the engineer’s discretion.

2.3 Method for predicting peak floor acceleration (PFA)

Similar to the peak IDR prediction method, the inelastic peak floor accelerations are computed using elastically-computed floor accelerations, and then modified to reflect the inelastic response. The elastic PFA profile, PFA_elastic in Figure 3, is computed using elastic response-history analysis with the first three modes of the structure. The elastic accelerations are then reduced to account for the inelastic effects and damping associated with the braces yielding. As the intensity of the ground motion increases, the difference in the elastic acceleration response and the inelastic acceleration response becomes greater, so there is a greater reduction of PFA as the strength ratio, S , increases. The amount of this reduction at each level is calibrated to the nonlinear dynamic analysis results. This PFA reduction is anchored at a story level, N_w , as shown in Figure 3, and then an acceleration profile shape is fit between the inelastic floor acceleration at level N_w and the PGA at the ground level. The accelerations above the level N_w are scaled up to reflect the observed “whip” in the upper levels of the building that is observed in the nonlinear dynamic analysis results. The story N_w is taken as the top 2 levels for buildings with 10 or more stories and the roof level for buildings with less than 10 stories.



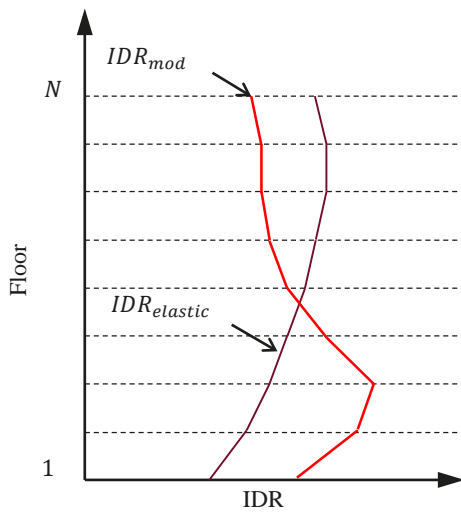


Figure 02: Peak interstory drift profile.

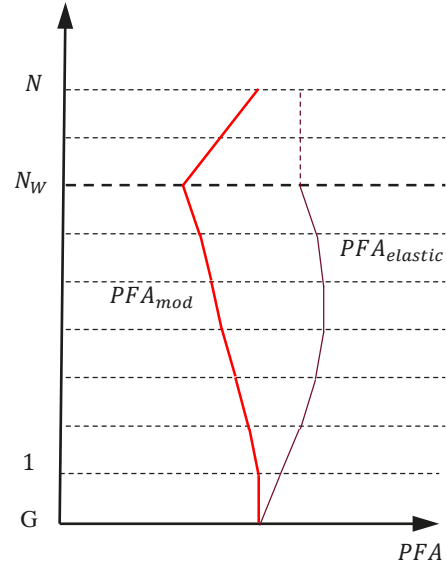


Figure 03: Peak floor acceleration profile.

2.4 Residual interstory drift model

The residual drifts experienced in the model were monitored for all different configurations. Based on the six variants considered in the analysis, three total residual drift models are proposed, as follows:

1. Model for CoreBrace BRBF with no backup frame, and no gravity frame contribution
2. Model for CoreBrace BRBF with no backup frame, but with gravity frame contribution
3. Model for CoreBrace BRBF with backup frame (with or without gravity frame contribution)

It is important to note that the gravity system and base connection variants (shown in Table 2) did not have a significant effect on the residual drifts for the structures with a backup frame, so no distinction is made in the proposed residual drift method. The data used to fit the data for the different residual IDR models are shown in Figure 4.

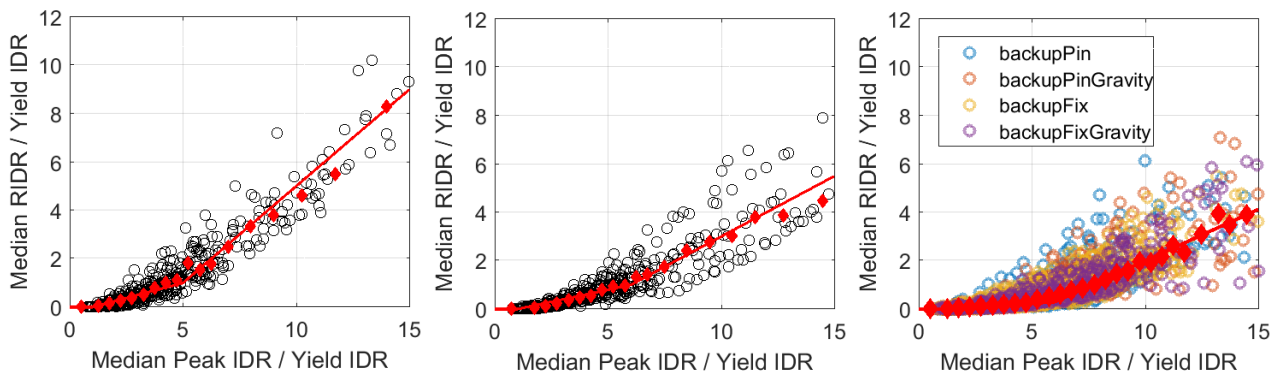


Figure 04: Residual drifts for frames with (left) no backup frame and no gravity, (center) no backup frame with gravity, (right) all configurations with backup frame



The three different residual drift models are shown in Figure 5. The results show that the residual drifts are not as severe as predicted using the FEMA P-58 [3] methodology, and improve considerably when considering the gravity system, and even more when a moment-resisting backup frame is included.

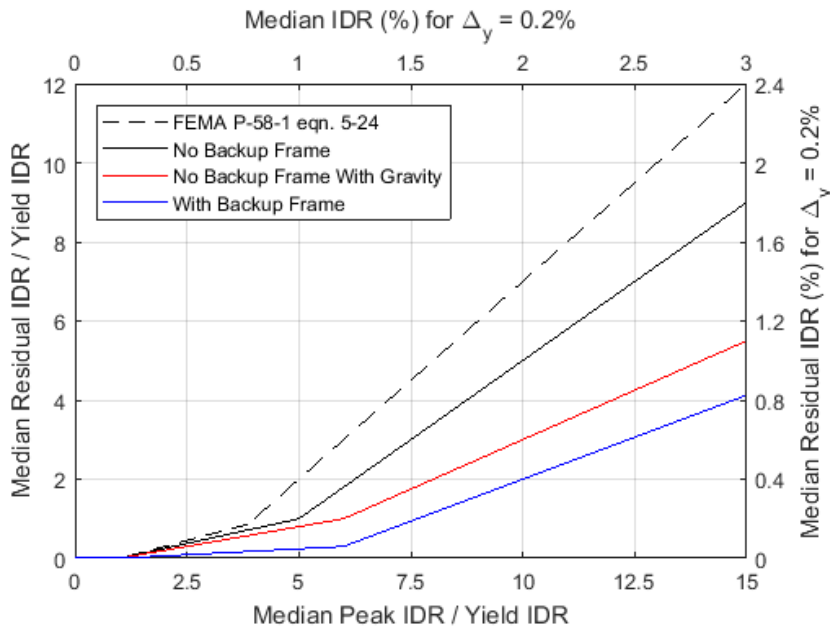


Figure 05: Residual drift model comparison.

3 SEISMIC RISK ASSESSMENT CASE STUDY

3.1 Site and structure definition

This new structural response prediction methods, most importantly the updated residual drift prediction method, were used to completed risk analyses of an example new 12-story BRBF building in Riverside, California, USA. This building was 30480mm square in plan with 6096mm single diagonal braced bays in each direction and a 3962mm story height. A floor plan is shown in Figure 6. The design, 5% damped, spectral response acceleration parameters at short periods and at one-second period of the site are $S_{DS} = 1.0g$, $S_{D1} = 0.6g$, respectively, and the mapped one-second spectral response acceleration parameter is $S_1 = 0.6g$. Required BRB capacities were determined based on an equivalent lateral force (ELF) analysis with a strength reduction factor, R , equal to 8.0.

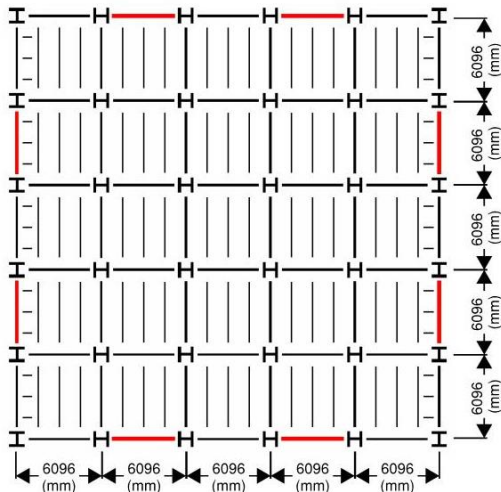


Figure 06: Plan view of 12-story case study structure.



3.2 Residual drift comparison

Figure 7 shows the results of these risk analyses, showing the loss curve predicted from the baseline FEMA P-58 residual drift method, as well as the loss curves computed with the updated residual drift models provided in this report. This shows that the updated residual drift models used in the loss assessment have a significant impact on the overall losses at both the 10% in 50 and 2% in 50 hazard levels. For reference, the loss ratios for Concentric Braced Frames (CBF) and Eccentric Braced Frames (EBF) have been computed and are also shown in Figure 7. The results indicate that all BRBF models perform at least as well as the CBF and EBF at the 10% in 50 hazard level. It would be necessary to engage the gravity system or to include a moment resisting backup frame of the BRBF system to equal the CBF and EBF at the 2% in 50 year hazard level, respectively.

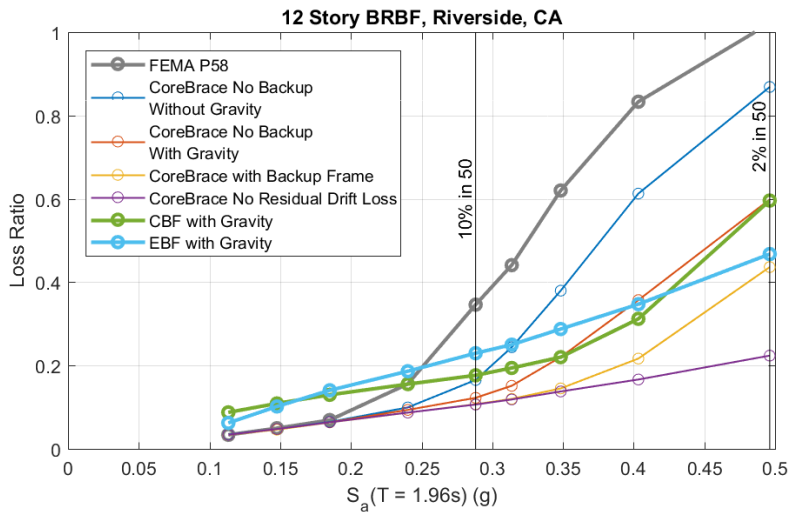


Figure 07: Loss curves for different residual drift models.

3.3 Probability of demolition due to excessive residual drifts

For the example building used in the case study, the median residual drift is shown in Figure 8, and the probability of demolition due to excessive residual drift, excluding the collapse cases, is shown in Figure 9. The probabilities shown in Figure 9 are based on using the median threshold for demolition due to excessive residual drift of 1.0% from the FEMA P-58 risk assessment method, for the peak residual drift at any level of the building. The results show that, with a back-up frame, the probability of demolition is nearly zero at the 10% in 50 year ground motion level. The model without a backup frame but considering the contribution of the gravity system is also nearly 0% and the model with neither backup frame nor gravity system is ~5%. All results are significantly better than the results predicted using the standard FEMA P-58 methodology, which predicts a nearly 20% probability of demolition at the 10% in 50 year ground motion and 70% at the 2% in 50 year ground motion. At the 2% in 50 year ground motion, the model considering the backup frame indicates only a 20% probability of demolition. Note that additional design steps could be taken to control residual drifts even at the 2% in 50 year level (MCE level).



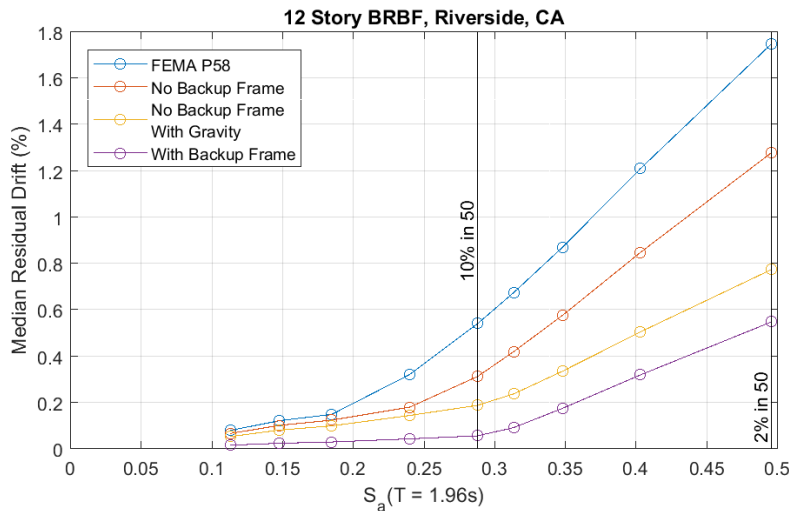


Figure 08: Median residual drift

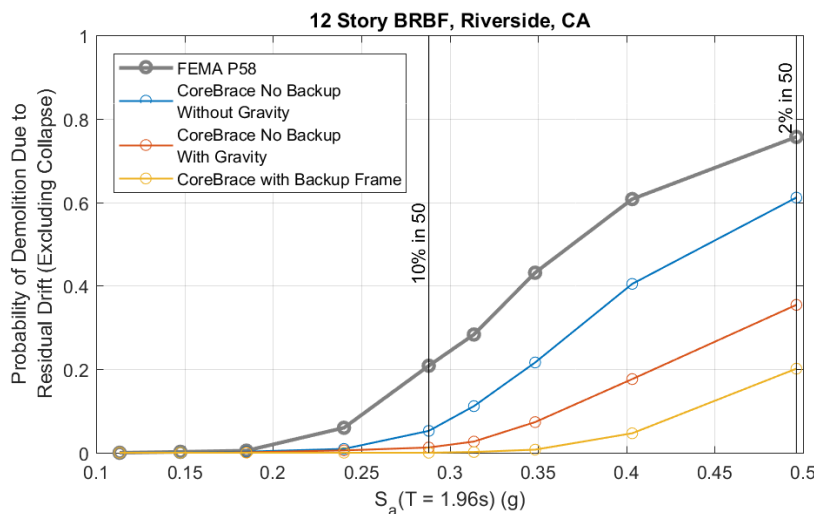


Figure 09: Probability of demolition due to excessive residual drift (excluding collapse cases)

4 RESILIENT DESIGN RECOMMENDATIONS

4.1 Resilient design methodology

The seismic risk assessment process outlined in the previous sections can be used for both assessment of existing BRBF building and for resilient design of new BRBF building. This section conceptually discusses how this approach can be used for resilient design, and specific resilient design guidelines are currently being created so that designers can complete a basic resilient BRBF design without needing to run the detailed FEMA P-58 analysis themselves.

A resilient design methodology requires ensuring the adequacy of both the structural and non-structural systems within the structure. This is often accomplished through checks of residual drifts (structural adequacy), drift sensitive items (such as partition walls and cladding), and acceleration sensitive items (such as equipment anchorages). A general outline to be considered when performing a resilient design is provided below.



1. Design for the greater of the design spectra and the 10% in 50 year intensity level (or at the hazard level for which resilient performance is desired). This ensures that the Building Code design process is not allowing the building to accept a higher risk in the design process (which is common for sites nearer to faults). In some cases it may be desirable to perform design at a higher intensity level (such as the 5% in 50) if resilient performance is desired at higher levels of ground motion or to meet higher performance levels.
2. Design for a limited level of structural damage, which does not require any repair before the building can be reoccupied and used as intended. This is fairly straightforward when utilizing BRBs due to their high strain capacities which can accommodate higher drift demands under the design level shaking. The column bases should also be designed and detailed to remain essentially elastic and not require any post-earthquake repair before the building can be reoccupied.
3. Ensure that peak drifts are low enough to prevent damage to drift-sensitive non-structural components. This is done by designing to a lower allowable story drift by including additional BRBF frames, or by increasing the capacity of existing frames.
 - a. In general, an interstory drift target of around 1% is typically needed to adequately protect the drift-sensitive non-structural components, and BRBF buildings often achieve drifts around this level simply based on the strength design process. This is especially true when designing with a response modification factor (structural ductility factor) less than 8.
4. Ensure that residual drifts are kept low. This can be accomplished with a combination of keeping the peak interstory drifts low (as addressed in the previous step) and possibly also including a moment resisting backup frame and/or a meaningful gravity system (which provides an elastic restoring force at low drifts, which provides a self-centering behavior). Note that past research has also shown the beneficial effects of a moment resisting back-up frame (with columns oriented in strong-axis bending) [4] as well as gravity column continuity [4].
5. Ensure that acceleration-sensitive components and anchorage are not damaged and do not require any repair before the building can be reoccupied and used as intended. This is made easier with BRBFs, as compared with other resilient design solutions that keep the structure quasi-elastic, because the nonlinear peak floor accelerations are kept lower due to the energy dissipation capability of the BRBFs. Lower floor accelerations, in combination with increased anchorage strengths and equipment capacities, are used to ensure that the equipment is not damaged.

As mentioned previously, this above resilient design process is currently being distilled into practical resilient design Guidelines for BRBF buildings, which do not require the user of the Guidelines to complete a FEMA P-58 seismic risk analysis, for general design practice. However to perform a thorough seismic risk analysis, the methods of FEMA P-58 or similar would need to be incorporated.

5 SUMMARY

The structural responses of a CoreBrace buckling restrained brace frame (BRBF) building were approximated based on a suite of nonlinear time history analysis for variants with and without a backup frame, considering and not considering the gravity frame contribution to the response. The residual drifts were also investigated and found to be significantly lower than the values predicted by FEMA P-58, especially when considering the contribution of a gravity system and lower yet when you include a backup frame. From these results, a case study in Riverside, CA shows that the losses as both the 10% in 50 and 2% in 50 hazard levels are significantly reduced by using the new residual drift models that are more specific to the behavior and performance of BRBF buildings. Additionally, the process of resilient design was outlined a simplified design methodology for resilient design was presented. A more complete design guide, based on the complete results of the study, is in progress and will be available in the near future.

6 REFERENCES

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