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Seismic performance of tall buildings designed following non-prescriptive design procedures

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ABSTRACT

Currently enforced building code requirements for earthquakes, including the guidelines for seismic design of tall buildings using non-prescriptive design procedures, are primarily intended to minimize life-safety risks due to structural damage under extreme earthquakes. While tall buildings designed under current standards are expected to achieve such life-safety goal, this study estimates their performance could require recovery times on the order of 3 to 9 months to repair damage from a design-level earthquake (roughly equivalent to ground motion shaking with a 10% chance of exceedance in 50 years). This study evaluates how recovery-based design guidelines may address these extensive downtime risks by enforcing: (i) tighter drift limits under expected ground motions; (ii) enhanced design criteria for critical nonstructural components; and (iii) measures to mitigate externalities that impede recovery. To illustrate these findings, a 42-story residential reinforced concrete shear wall building in San Francisco, CA is used as a case study. This paper is a summary of Part 3 of San Francisco's Tall Buildings Study, a recently completed Applied Technology Council Project for the City and County of San Francisco, in which the authors participated.

Keywords: Earthquake Engineering, Tall Buildings, Performance-based Seismic Design, Functional Recovery, Downtime.

INTRODUCTION

Currently enforced building code requirements for earthquakes, including the guidelines for the seismic design of tall buildings using non-prescriptive design procedures, are primarily intended to provide acceptable safety in extreme earthquakes. These design guidelines do not ensure performance beyond this life-safety objective, and do not limit damage to structural and nonstructural building components that could impact building functionality. As a result, even code conforming buildings, including non-prescriptive designs, may suffer significant damage from earthquakes. In addition to the costs to carry out repairs, damaged buildings may incur indirect losses due to extensive downtime.

In an effort to improve its seismic resilience, the City and County of San Francisco has put forward a set of post-earthquake recovery goals for its building and infrastructure [1]. For multi-family housing, the recovery target is to have 80-90% of the buildings regain their pre-earthquake functionality within "weeks." A similar recovery target is defined for major employers and hotels. Most of San Francisco's tall buildings fall within these categories, and pose an aggregate risk due to their high concentration in the City's downtown district. Furthermore, due to their size and high occupant loads, poor seismic performance in these buildings can have disproportionate impacts on the community.

This paper examines the performance expectation for new tall buildings and how it relates to building code requirements. A residential 42-story Reinforced Concrete Shear Wall (RCSW) building in San Francisco, CA is used as a case study. The building is designed following non-prescriptive procedures and its seismic performance is evaluated under code-level ground motions. The results of the evaluation are benchmarked against the City's recovery goals, and used to explore the potential costs and benefits of higher performance goals for new construction. A series of 'recovery-based design guidelines' are put forward to bridge the gap between the expected performance of these buildings and the City's recovery goals. This study is a subset of Part 3 of San Francisco's Tall Buildings Study [2], a recently completed Applied Technology Council project for the City and County of San Francisco, in which the authors participated.

CODE OBJECTIVES AND SEISMIC DESIGN REQUIREMENTS

San Francisco Building Code (SFBC) [3] requirements for earthquake design ensure acceptable safety in extreme earthquakes. These requirements are intended to limit risk of structural collapse, and damage to structural and nonstructural components that could pose life-safety risks under severe ground motions. Requirements to limit damage to components that could hinder building functionality or induce extensive recovery times are not currently considered in design.

Since 2008, buildings in San Francisco taller than ~73 m (240 ft) have been designed following the performance-based requirements of Administrative Bulletin 083 (AB-083) [4], *Requirements and Guidelines for the Seismic Design of New Tall Buildings using Non-Prescriptive Seismic-Design Procedures*. While these performance-based procedures help ensure reliability of building response under strong earthquakes, they are not intended to provide an enhanced level of performance, but rather are calibrated to provide equivalent performance to that of prescriptive designs.

The archetype RCSW residential building assessed in this study is designed following non-prescriptive procedures to meet the current seismic design requirements of the SFBC [3] and AB-083 [4]. The design process includes assessment of the building performance under three levels of ground motion shaking: a Service Level Earthquake (SLE), a Design Earthquake (DE), and a Maximum Considered Earthquake (MCE).

The SLE performance assessment is required to ensure an acceptable seismic performance (essentially elastic) under earthquakes that are anticipated during the service life of the building. The SLE level has a return period of 43 years (50% probability of exceedance in 30 years). The DE evaluation is intended to identify the exceptions taken to the prescriptive requirements of the SFBC [3] and to ensure the availability of minimum strength and stiffness for earthquake resistance. The DE level is defined as two-thirds of MCE-shaking. Finally, the MCE evaluation is used to ensure “collapse-prevention” under extreme ground motions and has a 2-5% probability of exceedance in 50 years. In contrast to linear analyses at SLE and DE, nonlinear response history analyses are employed in the MCE evaluation. Table 1 provides an overview of the design criteria adopted in the design of the archetype RCSW building. Detailed design requirements can be found in AB-083 [4] and PEER TBI [5, 6].

Table 1. Performance-based design criteria. Adapted from [2].

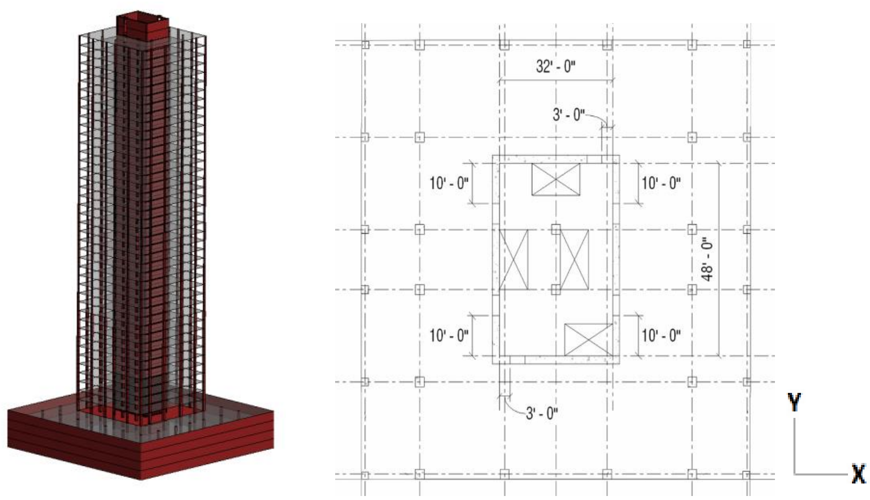
	Service Level Earthquake (SLE)	**Design Earthquake (DE)**	**Maximum Considered Earthquake (MCE)**
Objective:	Limited Damage	Code Compliance (Life-safety)	Collapse Prevention
Analysis Method:	Linear (Response Spectrum)	Linear (Response Spectrum)	Nonlinear Response History
Response Modification Factor (R):	---	Code-prescribed R factor	---
Story Drift Limits:	< 0.5%	< 2%	Mean Transient < 3% Mean Residual <1%
Component Acceptance Criteria:	Demand < 1.5 Nominal Strength*	Demand < Design Strength	Force and Deformation Controlled Component Checks
Guidance Document:	PEER TBI [6]	SFBC [3]	PEER TBI [6]

*In the previous edition of PEER TBI [5]: Demand < 1.5 Design Strength (strength reduction factors applied).

ARCHETYPE RESIDENTIAL REINFORCED CONCRETE SHEAR WALL BUILDING

The RCSW building evaluated in this study is a 42-story coupled core-wall residential building with a height of ~139 m (457 ft) above grade. The building was initially designed by Magnusson Klemencic Associates (MKA) for a site in Los Angeles [7], and later re-designed for a site in San Francisco by Tipler [8]. For this study, the building design is verified at two locations in San Francisco with distinct soil properties, a Site Class D (V_{s30}= 180-360 m/s or 600-1200 ft/s) and a Site Class B (V_{s30}= 760-1500 m/s or 2500-5000 ft/s) soil. Despite their distinct site class, designs are relatively consistent due to minimum base shear requirements at DE and other design constraints. A consistent design across sites with distinct seismic hazards is indicative of the variability in seismic performance that can occur across the City, and provides insights into the performance of buildings that have proportionally larger strength and stiffness compared to the ground shaking hazard, i.e. the building in Site Class D can be regarded as a baseline design versus an enhanced design in Site Class B. While this assumption serves the purpose of this study, in reality, design checks at SLE and MCE would result in final designs that would vary slightly across the two sites. In other words, the archetype building in Site Class B is slightly overdesigned.

The gravity system consists of post-tensioned 200 mm (8 in) thick slabs, measuring 32.9×32.6 m (108×107 ft) in the superstructure plan, supported by RC columns. The seismic force resisting system is a 9.8×14.6 m (32×48 ft) RC core consisting of coupled shear walls 810 mm (32 in) thick up to floor 13, and 610 mm (24 in) thick from floor 13 to the roof. The coupling beams are 760 mm (30 in) in depth in the superstructure and 860 mm (34 in) in the basement. Nominal compressive strength of concrete in the core is 55 MPa (8.0 ksi) and nominal yield strength of steel in the core and coupling beams are 410 MPa (60 ksi) and 520 MPa (75 ksi), respectively. Figure 1 shows an isometric view of the building and its superstructure plan view. The first two vibration periods of the building in each direction are summarized in Table 2. Further information on design details including reinforcement detailing in coupling beams and core walls can be found in [8].



(a) (b)

Figure 1. Archetype Reinforced Concrete Shear Wall building: (a) isometric view and (b) plan view. Adapted from [9].

Table 2. Dynamic vibration periods of the Reinforced Concrete Shear Wall building. Adapted from [8].

Period (s)	
Strong Direction (Y-axis)	Weak Direction (X-axis)
T_1=4.37	T_1=5.27
T_2=0.93	T_2=1.10

The RCSW building is designed to dissipate energy through hinging at the base of the wall piers and at the ends of the coupling beams up the height of the building. Flexural yielding of reinforcement is limited to the base of the walls where it is considered desirable by designing them for relatively high demand to capacity ratios. A 3D model of this structure, developed by [9] for the recently completed USGS HayWired study, is used to evaluate the performance of the building at DE and MCE levels by means of nonlinear response history analysis in LS-DYNA [10]. In this model, walls and coupling beams are modeled using distributed plasticity fiber beam-column elements and lumped plasticity elements, respectively. The hysteretic response of coupling beams is calibrated using the experimental data from testing at the University of California Los Angeles [11]. Columns and slabs are modeled using linear elastic beam-column and shell elements, respectively. More details on the modeling approach can be found in [8].

SEISMIC PERFORMANCE ASSESSMENT

Building response is evaluated at the DE intensity level. Two suites of eleven ground motions are selected and linearly scaled at the two locations of interest. As seen in Figure 2, the average spectra of the ground motion suites closely match the target spectra over the period range of interest (1 to 8 seconds). A detailed list of the selected ground motions can be found in [2].

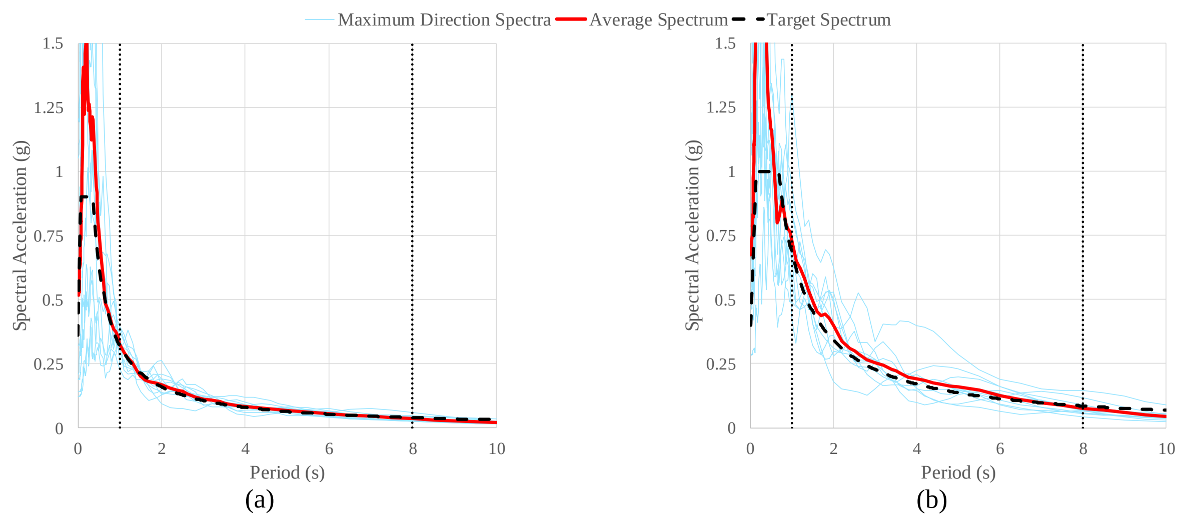


Figure 2. Design Earthquake target spectrum superimposed with the suite average and individual ground motions spectra: (a) Site Class B and (b) Site Class D.

Building performance is assessed using the FEMA P-58 framework [12], as implemented in SP3 [13], to evaluate repair costs and times. The REDi guidelines [14] are used to evaluate downtime to functional recovery. A building performance model, defined as a collection of structural and nonstructural building components susceptible to seismic damage, is developed for the assessment. Structural component quantities are estimated using the structural design information. Nonstructural component quantities are defined using the Normative Quantity Estimation Tool [12]. The resulting nonstructural component definitions are reviewed for consistency with previous studies on expected seismic performance of tall buildings similar to those under evaluation [7, 8]. Certain nonstructural components required the calculation of parameters to characterize their seismic resistance. Furthermore, user defined components are implemented to more accurately characterize damage to elevators and façade. Additional information on the user defined components can be found in [2].

A fragility function is assigned to each building component. Fragility functions are statistical distributions that indicate the conditional probability of incurring a damage state at a given value of demand. From each damage state, the associated repair costs and times are estimated by means of consequence functions. Inputs to the building performance model, defined as engineering demand parameters (EDPs), are obtained from nonlinear response history analysis. The following EDPs are used to evaluate performance in the RCSW residential building: story drifts, residual drifts, damageable wall drifts, racking drifts, coupling beam rotations and floor accelerations. Figure 3 shows the mean seismic demands up the building height in both building directions and sites of interest at the DE intensity level.

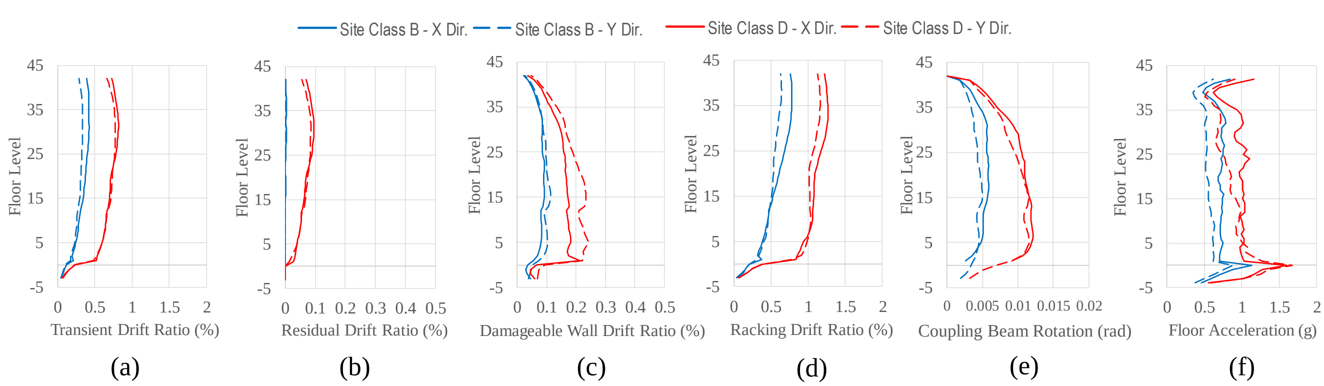


Figure 3. Mean demands from nonlinear response history analysis at the Design Earthquake: (a) transient drift, (b) residual drift, (c) damageable wall drift, (d) racking drift, (e) coupling beam rotation, (f) floor acceleration.

The downtime assessment methodology provides a framework to: (a) identify components that require repair to achieve a certain level of functionality, (b) develop a logical repair sequence on a floor-by-floor basis, and (c) estimate external factors that may delay the initiation of such repairs including post-earthquake inspection, engineering mobilization, contractor mobilization, financing, and permitting. These impeding factors are grouped into three delay sequences, the longest of which controls the impeding factor delay estimates. All delay sequences begin with post-earthquake inspection. Following post-earthquake inspection, the first sequence of delays considers the time to mobilize an engineer to carry out evaluations and any necessary design work, as well as delays associated with permitting such work. The second sequence considers the time associated with the mobilization of a contractor to carry out repair work. The last sequence relates to the financing of repair work. The total building downtime is the sum of the time required to repair components hindering a recovery state and the controlling sequence of impeding factor delays. Minor adjustments to REDi's downtime assessment methodology are implemented in this study, as outlined in [2]. Other factors, such as utility disruption or the time to procure specialty items (i.e. long lead components), are not considered in this study.

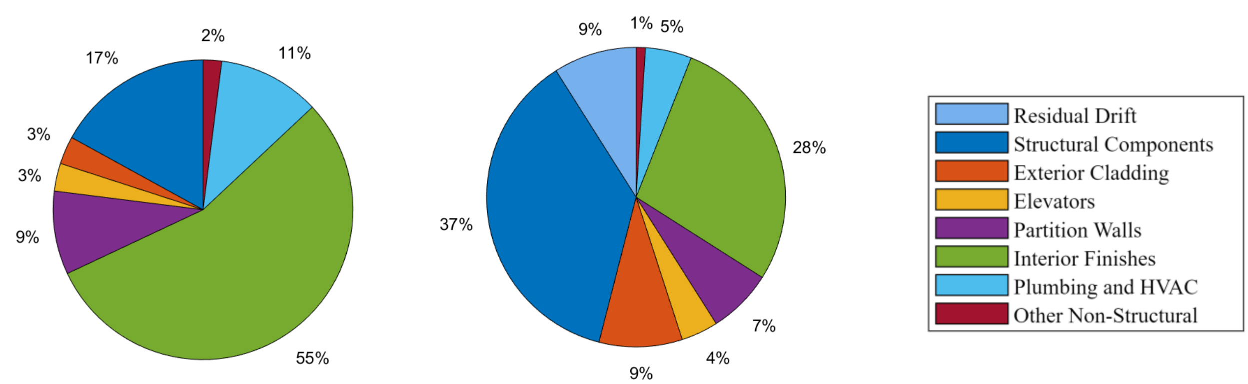
Monte Carlo simulations are carried out in the loss and downtime evaluation to capture the uncertainty in the performance. Median and 90th percentile results are reported in this paper. Table 3 provides a summary of the loss, repair time and downtime to functional recovery results. The loss results are normalized over the total building replacement cost, which is estimated at \$215M (\$312/ft^2).

Table 3. Summary of median and 90th percentile loss results, repair times and downtime to functional recovery with associated drifts at the Design Earthquake level.

Site Class	Drift* (%)	Loss (%)		Repair Time (days)		Downtime to Functional Recovery (days)	
		Median	90th Percentile	Median	90th Percentile	Median	90th Percentile
B	0.4 (0.8)	2.8	5.6	24	49	113	179
D	0.8 (1.3)	8.1	17.3	50	418	194	567

**Racking drift shown in parenthesis.*

The assessment methodology also enables identifying the contribution from different components to the loss, as illustrated in Figure 4 for the expected loss under the DE shaking. Site Class B losses are dominated by damage to interior finishes, whereas Site Class D losses are dominated by damage to structural components and interior finishes. Components that are sensitive to racking drifts, which are amplified by about 1.5 to 2 times over story drifts, drive the losses. Racking drifts are typically observed in core wall systems with perimeter gravity system due to differences in axial deformation (elongation) between the concrete walls and the gravity frame. The amplified racking causes damage and losses associated with interior partition wall finishes and slab-to-column connections.



(a) (b)

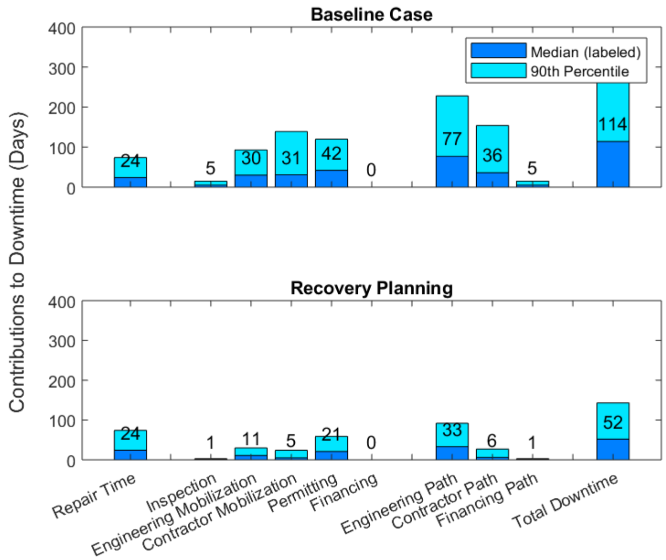
Figure 4. Contribution of different building components to the overall loss at the Design Earthquake: (a) Site Class B and (b) Site Class D.

TOWARDS RECOVERY-BASED DESIGN GUIDELINES

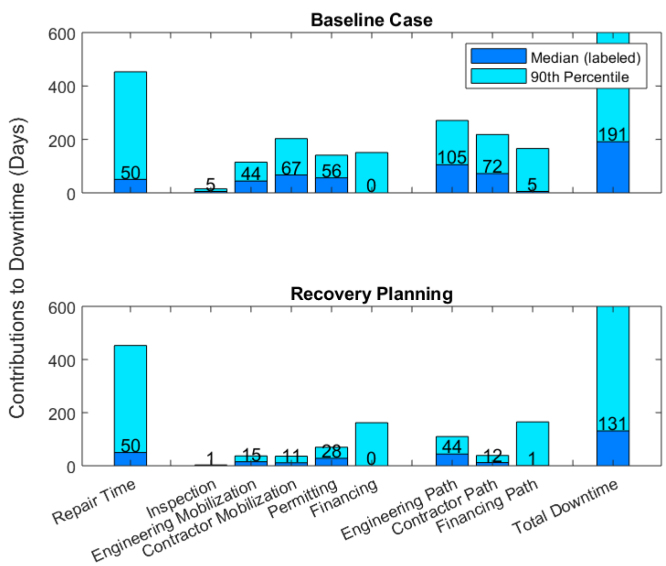
The differences in downtime to functional recovery between the baseline and enhanced designs suggest that stricter drift limits (larger strength and stiffness compared to the ground shaking hazard) can result in considerable reductions in downtime to functional recovery. However, as illustrated in Table 3, these measures alone are not sufficient to achieve the City's post-earthquake recovery goals. In order to further bridge the gap, a series of mitigation measures are explored. These include: (i) to expedite post-earthquake inspection, (ii) to have an engineer on contract to minimize delays associated with engineering mobilization, (iii) to have a general contractor on retainer to minimize delays associated with contractor mobilization, and (iv) to expedite permits for building repairs. Schemes (i) and (ii) already exist, through San Francisco's Building Occupancy

Resumption Program (BORP) [15], and arrangements that building owners frequently hold with engineers to carry out certain activities as needed. Schemes (iii) and (iv), while not routinely available, are evaluated to explore their impact on expediting recovery.

De-aggregating the contributions of different factors towards downtime to functional recovery is not as straightforward as de-aggregating seismic losses. De-aggregation of downtime to functional recovery, illustrating the median and 90th percentile estimates of each downtime contributor, is shown in Figure 5 for the baseline and enhanced buildings with and without recovery planning. For the baseline design, mitigation measures can help reduce downtime to functional recovery from approximately 6-1/2 months (191 days) to just over 4 months (131 days). For the enhanced design, mitigation measures can help reduce downtime to functional recovery from approximately 4 months (114 days) to just over 1-1/2 months (52 days).



(a)



(b)

Figure 5. Downtime de-aggregation at the Design Earthquake with and without recovery planning: (a) Site Class B and (b) Site Class D.

CONCLUSIONS

This study evaluates the seismic performance of an archetype 42-story RCSW residential building in San Francisco, CA. The evaluation is intended to benchmark expected performance against the City's post-earthquake recovery goals, which indicate the majority of multi-family residential buildings should regain pre-earthquake functionality within "weeks" of a major earthquake. The findings of this study suggest that to achieve this goal, recovery-based seismic design guidelines are required. Such requirements should include: (i) tighter drift limits under expected ground motions, (ii) enhanced design criteria for critical nonstructural components, and (iii) measures to mitigate externalities that impede recovery.

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