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ATC-145: FRAMEWORK FOR ASSESSING THE EXTENT OF REPAIR REQUIRED FOR EARTHQUAKE-DAMAGED CONCRETE MOMENT FRAMES

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SUMMARY

ATC-145 is a FEMA funded project aimed at developing a Guide for repair of earthquake damaged buildings to achieve future resilience. Thanks in part to EQC funding, the project technical committee includes substantial New Zealand representation.

The first two years of ATC-145 have focussed on when minor repairs (i.e. epoxy injection, patch repair of spalling, and similar simple repairs) are sufficient to ensure adequate future performance of earthquake-damaged ductile concrete moment frames and determining thresholds beyond which minor repair is insufficient. The overall framework considered necessary to answer this question comprises phases focussed on inspection, analysis, and separate assessments of impact on safety and serviceability. Subsequent parts of the project are focussed on case studies to validate the framework, repair guidelines, and extending the approach to other structural systems.

A practitioner-focussed overview of the phases involved in the overall framework, including discussion of the performance objectives and the rationale behind the selection of these objectives is contained in this paper.

INTRODUCTION

The Federal Emergency Management Agency (FEMA) is funding an ongoing project focussed on *Development of Guidance for Repair of Earthquake Damaged Buildings to Achieve Future Resilience*, which is managed by the Applied Technology Council (ATC). The ATC project reference, ATC-145, is used throughout this paper as shorthand for the project. To date the project has focussed on identifying when simple repairs (i.e. epoxy injection, patch repair of spalling, and similar repairs) are sufficient to ensure adequate future performance of earthquake-damaged ductile concrete frames (i.e. frames able to sustain a displacement ductility of at least μ = 3 and expected to form a beam sway mechanism).The draft assessment framework developed to guide the process of inspecting and assessing earthquake damaged frames is summarised in Figure 1. The assessment framework involves three phases:

- Inspection and analysis phase
- Safety-assessment phase, and
- Serviceability-assessment phase

The three phases of assessment are described in more detail in the body of this paper, but in summary they have goals as follow:

- The Inspection and Analysis phase aims to identify any "severe damage states" in the structure which clearly require complex repairs and to estimate the peak drift demands from the damaging earthquake.
- The goal of the safety-assessment phase is to determine if the damaged building and/or component requires complex repairs to be able to satisfy a post-earthquake safety performance objective, including minimum criteria prescribed by the IEBC, i.e. the International Existing Building Code (ICC 2018)
- If the building and/or component satisfies the safety-assessment phase, it may also be necessary to undertake the serviceability-assessment phase to determine if damage to drift-sensitive non-structural components is expected in future low intensity earthquakes.

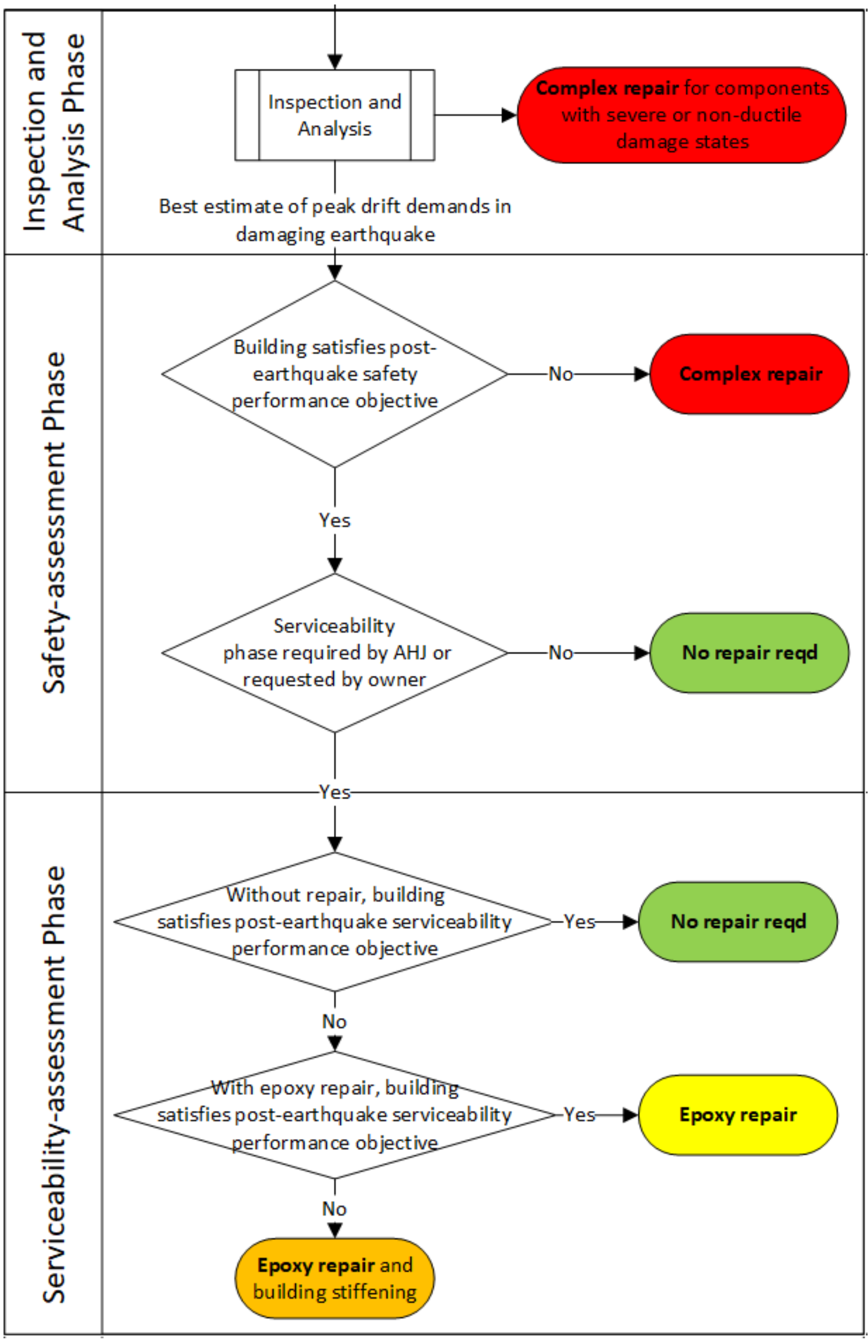

Figure 1: Assessment framework flowchart

The assessment framework is not intended or appropriate for a rapid post-earthquake assessment, including placarding or tagging operations; rather, it is intended for a detailed assessment which is likely to take place in the weeks or months following an earthquake.

It is emphasised that the assessment framework has, to date, focussed on the behaviour of ductile moment resisting frames. It is not directly applicable to other structural systems. It also, due to its U.S.-centric basis, does not consider the implications of an earthquake to any precast concrete floors that may be present. These, and other New Zealand specific aspects require further consideration.

PERFORMANCE OBJECTIVES

To determine whether structural repairs are required it is necessary to identify performance objectives for the damaged building being considered.

ATC-145 defines two performance objectives, namely:

1. Safety criterion: Assessed performance of building under design basis earthquake (DBE) and maximum-considered earthquake (MCE) ground motions remains 'essentially unchanged after the damaging earthquake.
2. Serviceability criterion: Drift demands for a "service earthquake" do not exceed the median drift capacity of drift-sensitive non-structural components present in the building (e.g., partition walls) after the damaging earthquake.

The safety criterion effectively implies that a building that could withstand two repeated ground motions while still satisfying the performance objective of U.S. codes (e.g., less than 10% probability of collapse in MCE) would not require repair.

For ATC-145, a building is assumed to satisfy the safety criterion when the median drift demand, assessed at DBE and MCE levels, remains "essentially unchanged" after the damaging earthquake compared to that experienced by an undamaged building, and the deformation capacity of structural components have not been compromised. The definition of 'essentially unchanged' is encompassed within the repair triggers defined for the project.

Additionally, the International Existing Building Code has capacity-based triggers for repair and strengthening that may be invoked by the relevant authorities. It is anticipated that the ATC-145 safety-assessment phase will provide results for use with these provisions. Specifically, IEBC requires that:

- 'Compliant' buildings (75% of the capacity of a new building, or specified performance level based on assessment procedures) are required to be repaired to their pre-damage condition. This requirement also applies to 'non-compliant' buildings suffering insubstantial damage.
- 'Non-compliant' buildings that have sustained 'substantial structural damage' must be repaired and retrofitted so that they become 'compliant'.

Substantial structural damage is defined as a 33% reduction of lateral capacity at any storey or a 20% reduction of gravity capacity of vertical elements that support more than 30% of the structure and where the residual vertical capacity is less than 75% of that required for a new building.

The definition of the serviceability criterion has arisen because, in contrast to New Zealand practice, U.S. design practice for typical structures does not explicitly consider the serviceability limit state (SLS). Consequently there is no formal U.S. building code definition of a 'service earthquake'. The criterion proposed is essentially consistent with New Zealand design approaches for SLS and is considered a reasonable basis for assessments in New

Zealand, where the 'service earthquake' would be defined as the SLS earthquake for the building.

INSPECTION AND ANALYSIS PHASE

As the name suggests, the Inspection and Analysis phase includes inspection and analysis of the damaged building. The purpose of this phase is twofold:

1. Identifying any severe damage, noting however that initial inspections are likely to have been undertaken previously whether for placarding or other purposes, and
2. Estimating the peak drift demand experienced by the building in the earthquake.

Figure 2 provides a flowchart for the steps envisaged. The feedback type approach whereby inspection informs analysis and vice-versa will be familiar to many New Zealand engineers who investigated damaged buildings using similar approaches during the last decade.

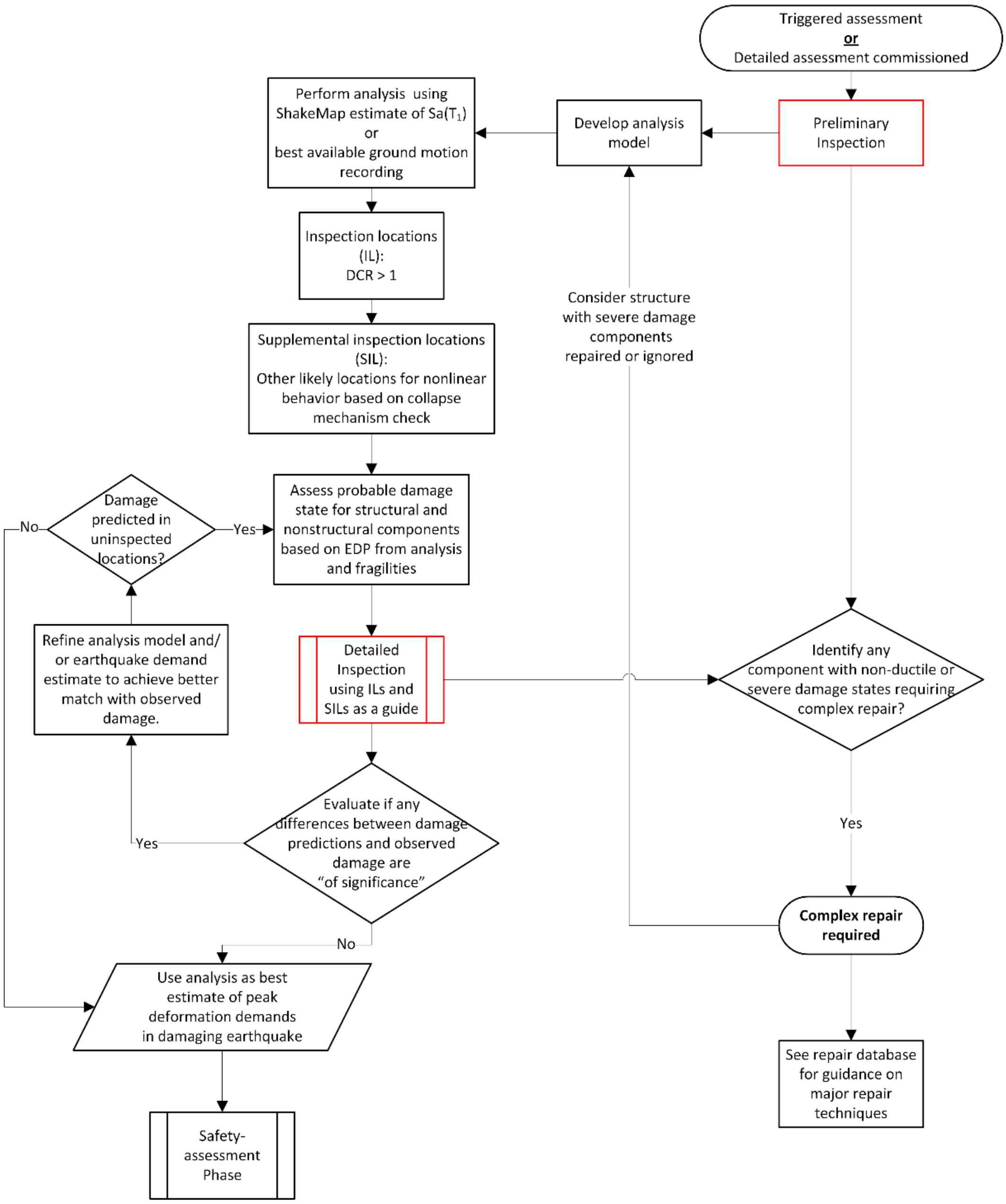

Figure 2: Inspection and Analysis phase

In (currently) rare instances, buildings may be equipped with instrumentation that can be used to infer peak response incurred during the earthquake, this data has obvious value during both inspections and analysis. Locations of maximum deformation or story-drift demand should be identified for visual inspection. Analysis of the building can be calibrated with the recorded response over the building height (e.g. Siddiqui et al. 2019), and if the instrumentation also reports accelerations at the base of building, the resulting response spectrum or ground motion may be used as a direct input to the analysis model.

Inspections

Site inspections, whether preliminary or otherwise have two foci, namely identification of severe damage that obviously necessitates ‘complex’ repair (e.g. component replacement), and identification of structural or non-structural damage that can serve as indicators of the likely drift imposed on the building during the earthquake.

The overall assessment framework is predicated on use of a repair trigger (imposed drift or deformation) to determine whether complex repairs are required or not. However, it is acknowledged that the observation of severe, non-ductile or force-controlled damage types would inevitably necessitate significant repair, irrespective of the inferred deformation demand. These damage types include (but are not limited to):

- Concrete core crushing
- Shear failures
- Spalling with evidence of bar buckling or fracture
- Tearing or damage to floor diaphragms at points of high stress (e.g., re-entrant corners, perimeter column to slab connections).
- Significant damage to precast concrete floors
- Large residual story drifts or foundation settlement

In buildings that are deemed potentially repairable despite some degree of severe/non-ductile damage or failure being observed during the preliminary inspection, the ATC-145 process may still be applicable to define the residual capacity of the remaining portions of the building not exhibiting such damage. For example, if a building exhibited non-ductile diaphragm damage that is deemed repairable, this procedure could still be applicable to the lateral force resisting system. Alternatively, if the component experiencing severe/non-ductile damage is sufficiently damaged as to not to participate in the building response and can be ignored without loss of stability, the assessing engineer may decide to ignore the contribution of the failed component in the remainder of the assessment process. Judgment must be used as to when the observed non-ductile damage prevents this procedure from being reasonable to apply.

For the purposes of the ATC-145 assessment framework, inspections should also focus on observation of damage that can serve as a proxy to infer peak drift demands from the earthquake. For example, relevant non-structural damage includes sliding/movement of joints, damage to gypsum sheathed wall partitions, and pounding (interaction) of secondary structural systems (e.g., stairs) with primary structural elements (walls or columns).

Analysis of the building

Analysis of the earthquake-damaged building requires decisions to be made regarding:

1. The nature of the analysis undertaken, and
2. The representation of the earthquake shaking imposed on the building.

ATC-145 recommends that analysis should follow one of the methods described in ASCE 41 (2017), namely linear static (LSP), linear dynamic (LDP), non-linear static (NSP), or non-linear dynamic (NDP). It is recommended that a linear procedure generally be used as a starting point, with a modal analysis LDP being the preferred option except in rare cases where a ground motion recording is available at the building site when a response history LDP may be an appropriate alternative. Only in cases of irregular buildings or those with considerable structural damage observed in the preliminary inspection are non-linear procedures warranted for the initial analysis.

Preliminary Inspections should inform the development of the analysis model. If significant or extensive evidence of degradation or yielding (e.g., beam hinging) are observed during the Preliminary Inspection, a linear analysis model may not be appropriate, and it may be advisable to proceed directly to a nonlinear analysis.

Analysis requirements that are intended to produce a conservative value of deformation or force demands should be treated with caution, as the goal of the analysis for this post-earthquake assessment procedure is to obtain a best estimate of the actual deformation demands incurred. For example, torsional amplification requirements will generally overstate the mass eccentricity and thus artificially increase demands.

The representation of seismic demand used in the analysis is intended to simulate, as closely as is practically possible, the ground shaking experienced by the building. Excepting the rare cases where a site-specific recording station is available, an estimate of site-specific ground shaking will include uncertainty due to attenuation and/or spatial interpolation and differences in ground conditions between building site and recording station sites:

- Currently this is likely to require consideration of available shaking records from nearby strong motion stations and selection of the most appropriate record (i.e. without significant differences in terms of site conditions) or interpolation between a number of relevant stations. Where available, advantage should be taken of published resources that can guide the interpolation (e.g. Bradley and Hughes 2012; Canterbury Geotechnical Database 2013; Worden et al. 2018)
- Increasingly in the future, it may be possible to use ground motion simulations to estimate the ground motion at any site. For example, SeisFinder (Bradley et al. 2017a; Savarimuthu et al. 2017) is a web application to extract selected intensity measures from previously conducted ground motion simulations of historical events. Ground motion simulation technology is rapidly evolving and may be in widespread use in the future, though careful review of validation with available recordings is essential (Bradley et al. 2017b).

Care is required to ensure that the predicted building response is not unduly influenced by spectral peaks and troughs present in particular earthquake recordings that may not be characteristic of the site of the building. This is likely to be a particular consideration when using records from particular recording stations, whereas published interpolation data such as ShakeMap (Worden et al. 2018) generally contains inherent smoothing of such effects.

<u>Estimate of deformation demand</u>

After completion of initial analyses, an iterative sequence of further, targeted, damage inspections and refinement of the analysis is envisaged, with this process continuing until the engineer is satisfied that the deformations predicted by the analysis are reasonably consistent with the observed condition of the building.

Two parameters describing the deformation response of the building to the damaging earthquake are required for the assessment phases, namely:

- The peak story drift, δ_{EQ}, and
- The peak chord rotation, θ_{EQ}.

Story drift is recommended as the deformation demand metric of choice due to its widespread use and familiarity in earthquake engineering. However, for ductile hinging elements the chord or plastic rotation may be better correlated with the degree of structural damage observed.

To develop confidence in the analysis it is necessary to confirm that predicted deformations correlate reasonably with the damage observed in the building. Quantitative relationships between observable damage and deformation demand would be optimal tools for this purpose. Tools for developing quantitative estimates are discussed subsequently; however, the uncertainty involved in such an estimation means that even a notionally quantitative procedure would involve significant subjectivity. The experience and judgement of the assessing engineer is therefore perhaps the most important factor in developing such an estimate. Review of the assessment by an independent, experienced engineer is recommended.

Perhaps the most comprehensive source of data that explicitly relates deformation demand to observable damage is the fragility curves developed as part of FEMA P-58 (Applied Technology Council 2019). Using the estimate of deformation demand for the specific component being considered, it is straightforward to use the fragility functions to determine probabilities of being in various damage states (e.g. Figure 3). Non-structural fragilities are likely to be beneficial, as they can exhibit tighter ranges of drifts between visually distinct damage states. For example, the difference in damage observed in a ductile reinforced concrete plastic hinge subject to 0.5% or 1.5% drift may be limited, while it may have a significant effect on partition walls.

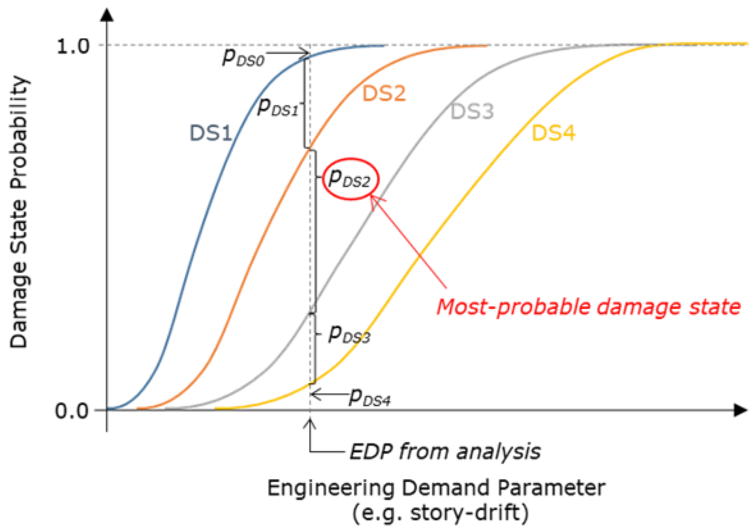

Figure 3: Use of fragility curves to identify probable damage state.

Other published relationships between deformation demand and observable damage exist, for example for walls and coupling beams (FEMA 1998) and columns (Bearman 2012). These are considered supplementary sources to FEMA P-58 and may enable more refined estimates of expected damage to be identified.

Caution must be exercised if post-earthquake (i.e. residual) crack widths are used as a metric. Tests conducted by Marder et al. (2020b) showed that crack widths are dependent on the level of axial load or axial restraint and the residual deformation, but not necessarily the peak deformation. Alternative metrics, such as the number cracks or distribution of cracking along a plastic hinge, may be better correlated with peak demands.

Further inspections are targeted at:

- Inspection Locations (IL), defined as regions subject to large demands (currently specified as any structural component identified by the building analysis where the strength Demand Capacity Ratio (DCR) exceeds 1.0, but with this subject to change) and

- Supplemental Inspection Locations (SIL), defined as any structural component that is not already identified by the building analysis, but where nonlinear behaviour is anticipated based on a frame-mechanism check or an observed structural configuration irregularity.

Broadly, inspections would be expected to comprise visual inspection of ILs and SILs, necessitating exposure of the surface of the structural component. Cover concrete should not be removed unless already spalled or loose. If the cover concrete is undamaged, the core and bars can be assumed to be undamaged. A sampling process may be adopted to potentially reduce the number of inspections, but this has yet to be defined. Further refinement and consideration is also required regarding the appropriateness of assuming that a DCR exceeding 1.0 is a reasonable threshold for the onset of damage in a ductile moment frame.

Material sampling and testing is not required as part of the assessment framework.

No prescriptive rules are proposed regarding what constitutes acceptable agreement between observed and estimated damage. The engineer must exercise their judgment based on the situation. The following three possible scenarios are highlighted as warranting consideration:

1. The analysis results give deformation demands large enough to trigger the limiting criteria in the safety-assessment phase, but the detailed inspection shows limited damage.
2. The analysis results give deformation demands small enough to not trigger any of the limiting criteria in the safety-assessment phase, but the detailed inspection shows considerable damage.
3. The distribution of damage observed in the detailed inspection does not align with the expected mechanism based on the analysis.

Of these three scenarios, the first is assumed to be the most likely to occur, due to the conservative nature of the seismic assessment provisions on which the analysis is based. The other two scenarios likely require refinement of the analysis or verification of input assumptions. If none of these three scenarios is observed, there is little rationale in attempting to refine the analysis as it would potentially have no effect on the overall outcome of the assessment procedure.

The analysis refinement process is intended to reduce errors between analysis results and inspection findings. However, it must be emphasized that the reduction of “error” is not in itself the goal. The goal is to better represent the true seismic response of the building during the damaging earthquake, and thereby better understand the future seismic performance of the building. Any number of modifications to the analysis model or input demands could be made to reduce the discrepancy, but they should only be considered if they are likely to influence the outcome of the overall assessment (i.e., the need to repair).

The logical areas for analysis refinement to investigate depend on the discrepancies observed:

- Misalignment between observed and predicted damage locations may indicate non-linear analysis as an appropriate way of better understanding the post-yield response of the building. In particular, buildings observed to have exhibited localized damage (e.g., soft storey mechanism) may be poorly predicted by linear analysis.
- If the deformation demands obtained from the analysis imply a higher level of damage not consistent with the damage observed in the inspection, conservatism in the modelling is likely to be a contributing factor.
- If the deformation demands obtained from the analysis imply a lower level of damage than the damage observed in the inspection, an understated representation of seismic demand (i.e. ground motion estimate) may be the most probable cause. This is based on the assumption that typical modelling criteria often err on the conservative side and thus are less likely to be a contributing factor to this type of discrepancy.

- Discrepancies between the as-built conditions and the drawings used as input for the analysis model may also lead to differences in the expected and observed performance. The engineer should take reasonable steps during the inspection to confirm that the as-built conditions match the drawings.

SAFETY ASSESMENT PHASE

The goal of the safety-assessment phase is to determine if the prior earthquake demands on a building exceed a "repair trigger", thus requiring complex repairs in order to satisfy the post-earthquake safety performance objective. Such repair may be triggered if an amplification in drift demands in a design-level earthquake ground motion is anticipated and the prior loading has resulted in a reduction of component deformation capacity. The assessment is undertaken based on the peak story drift, δ_{EQ}, and peak chord rotations, θ_{EQ}, from the prior earthquake demands as estimated during the Inspection and Analysis phase.

The series of checks undertaken during the safety assessment phase is shown in Figure 4. These comprise checks on the maximum drift sustained by the building ('system check' – purple diamond), the maximum chord rotation imposed on beams ('component check' – blue diamond), and the extent of fatigue demand on longitudinal reinforcement ('fatigue check').

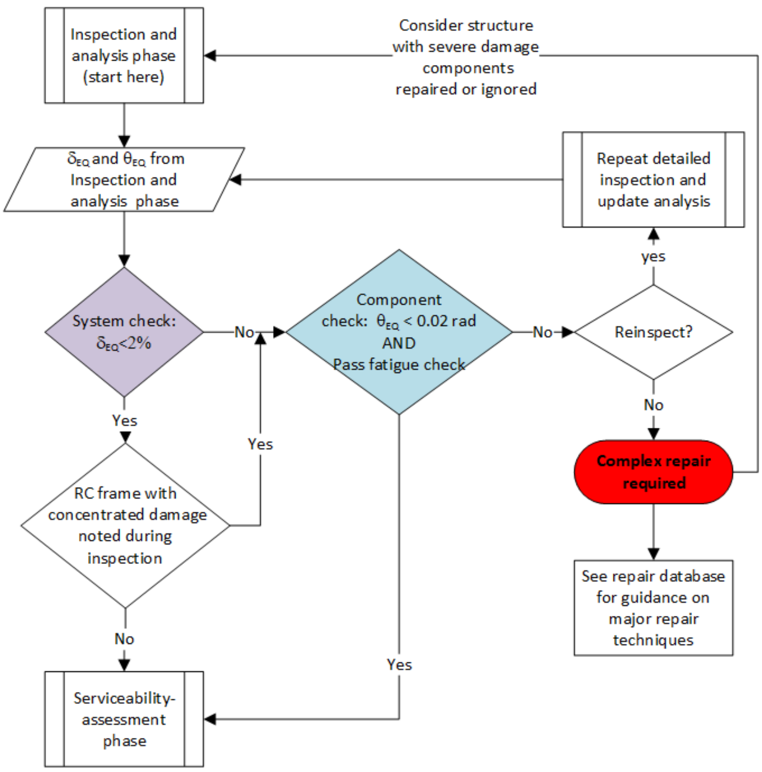

Figure 4: Safety-assessment phase

The checks were developed from studies undertaken for the ATC-145 project comprising:

- System (i.e. whole structure) analyses to determine when amplification in drift response in a repeated earthquake is anticipated, along with review of previous experimental work applicable to the same question.
- Assessment of RC frame component tests with varying loading protocols to determine when prior loading will result in decrease in deformation capacity of the component,
- Analytical assessment of conditions under which fatigue capacity of reinforcing bars may have been reduced such that fracture in a repeated earthquake is anticipated.

Further detail regarding these studies is provided in Elwood et al. (2021).

The results of the studies support the conclusions that:

1. Amplification in drift demand in a repeated design-level ground motion is not anticipated if the peak storey drift, δ_{EQ}, does not exceed 2%. For many frame structures, this criterion alone should be sufficient to ensure the building satisfies the post-earthquake safety performance objective;
2. If the building does not pass the above criterion, or if concentrated damage was noted during detailed inspection, a component-level check should be completed. Generally, if the peak chord rotation for a component, θ_{EQ}, is less than 0.02 rad then it is unlikely that the prior loading has detrimentally impacted the drift capacity of the component

The chord rotation check was developed considering a broad range of loading protocols but does not directly address fatigue of reinforcement. If spalling of concrete is not observed then fatigue of reinforcement can be dismissed as a concern. Where spalling sufficient to expose a length of longitudinal reinforcement in excess of 2 bar diameters is evident, a fatigue check is required to ensure the reinforcement is unlikely to be materially degraded. The fatigue check is considered satisfied provided:

1. Bars have not visibly buckled, and
2. The maximum chord rotation is less than 0.02 rad, and
3. The damaging earthquake is not of unusually long duration[1], and
4. The effective plastic hinge length defined on a typical basis (NZSEE et al. 2018) of an element is greater than 0.4 times the member depth.

The fatigue criteria above have been developed assuming that reinforcement is not susceptible to strain ageing as is the case for common U.S. reinforcement and for New Zealand microalloy Grade 500E reinforcement (Pussegoda 1978; Restrepo-Posada 1993). Additional work is required to cover situations where longitudinal reinforcement is susceptible to strain ageing such as when mild steel (e.g. Grade 300) reinforcement is used (Loporcaro et al. 2016).

SERVICEABILITY ASSESSMENT PHASE

After satisfying the safety-assessment phase, the building is deemed to meet the post-earthquake safety performance objective. It is noted that the safety-assessment phase does not identify any need for category 1 simple repairs (namely, epoxy injection). This is because epoxy injection has been found to have limited impact on the ultimate deformation capacity, and hence safety-related performance, of ductile reinforced concrete components. Hence, epoxy-injection repair is only considered useful for increasing stiffness and reducing drifts for service-level earthquakes.

This section overviews a process for identifying if epoxy injection, or further intervention, is required to meet the serviceability post-earthquake performance objective. In some jurisdictions the serviceability phase may not be required, in which case a building not requiring complex repair from the safety-assessment phase may be left unrepaired from an earthquake safety perspective. However, considerations such as durability, fire protection, or aesthetics may lead to an epoxy (or other) repair of damaged components regardless of the choice to consider the serviceability-assessment phase.

Damaged buildings are generally expected to experience a larger drift in a service earthquake than an undamaged building because the response of both the damaged and undamaged buildings due to a small service-level earthquake are expected to be essentially linear-elastic, but the damaged structure will be softer than the undamaged structure. However, such amplification of displacements is only a concern for a service earthquake if the amplification

[1] As a placeholder, the limit for a ‘normal’ duration earthquake is set as a significant duration ($D_{5\text{-}95}$) of less than 45 sec Further investigation is required to confirm the appropriateness of this value.

results in non-structural damage or occupant discomfort in a service earthquake not experienced in the building prior to the damaging earthquake (i.e., for an undamaged building). Wind loading may also be a consideration, particularly for taller buildings.

Since the damaged building will be softer than the original building, it is expected that drift-sensitive non-structural components, rather than acceleration-sensitive components, will be most impacted. It is, thus, recommended that the engineer compare drift demands from a linear analysis (using appropriately reduced stiffnesses) with the drift capacity of the most critical drift sensitive component in the structure. This is anticipated to be either cladding or partition walls. The objective is to ensure that the structure stiffness has not reduced so much as to lead to repeated damage to such non-structural components during (relatively frequent) service earthquakes. It is noted that low-level earthquakes are more frequent during the months and years following a major earthquake until the aftershock rate decays to steady-state seismic activity, and repeated damage to recently repaired non-structural components during aftershocks is not desirable.

As summarised in Figure 5, the serviceability assessment phase is relatively simpler than the earlier phases of the framework. The goal of this phase is to identify if the drift demands in a service-level earthquake, δ_{SLE}, are likely to exceed the drift capacity of critical drift-sensitive non-structural components, δ_{NS}. The first step is to assess if the damaged building can satisfy $\delta_{SLE} < \delta_{NS}$ without any repair. If the damaged building does not meet this criterion, repair will be necessary. If drift demands using the stiffness of epoxy-repaired components is still not sufficient to satisfy $\delta_{SLE} < \delta_{NS}$, either the building will need to be stiffened or the critical non-structural components will need to be upgraded to increase δ_{NS} so that they are able to accommodate δ_{SLE}.

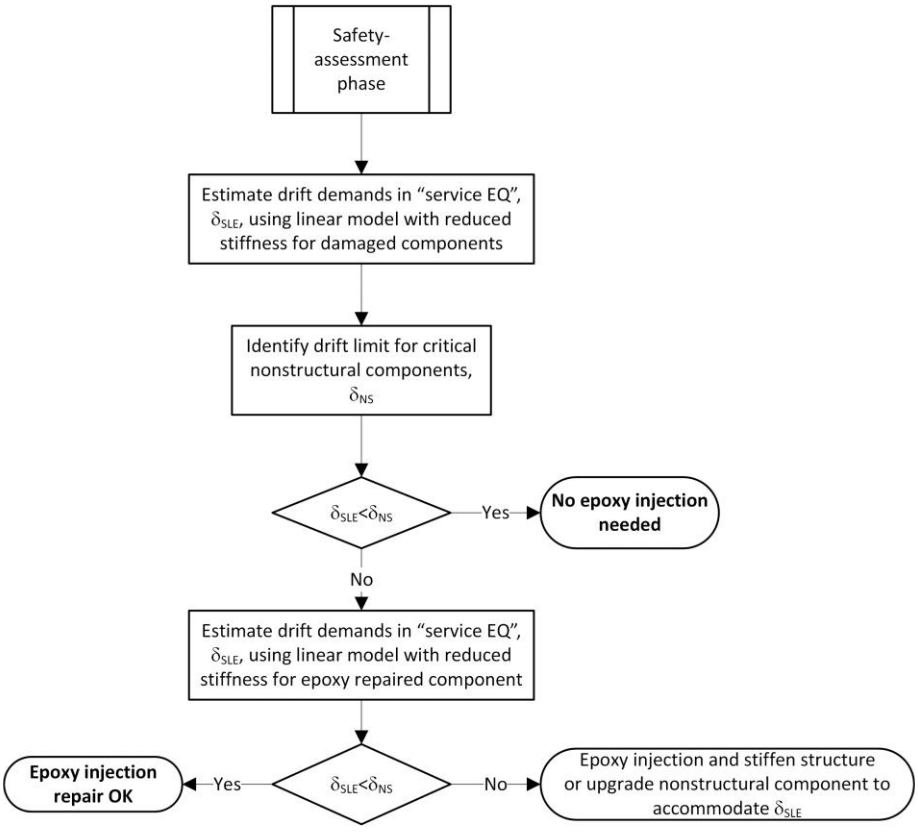

Figure 5: Serviceability-assessment phase

Stiffness of damaged and repaired components

RC frame components will decay in stiffness with an increase in prior inelastic drift demand. Gulkan and Sozen (1974) suggested that the reduced stiffness of a component having experienced displacement ductility of μ could be estimated based on the origin-to-peak stiffness as shown in Figure 6. Marder et al (2020b) indicate that this model generally provides a reasonable lower bound to the measured stiffnesses of columns from repeated shake table tests, except for ductility demands less than 2. To account for this, and given uncertainties in estimating the prior ductility demand they recommended to determine the reduced stiffness as:

$$\frac{K_r}{K_y} = \begin{cases} 1.0, & \mu < 1.0 \\ 0.5, & 1.0 \le \mu \le 2.0 \\ 1/\mu, & \mu \ge 2.0 \end{cases} \tag{1}$$

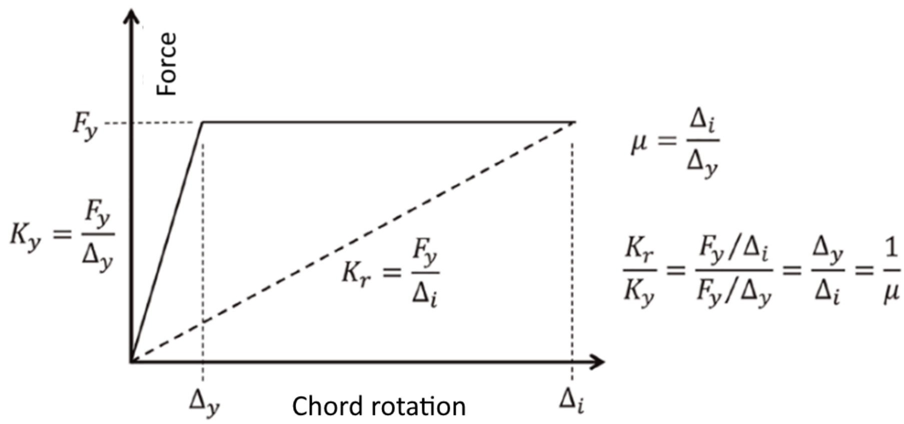

Figure 6: Origin-to-peak stiffness.

Epoxy-repair can recover a portion of the stiffness loss due to prior loading. The extent of stiffness recovery depends on quality control and the ability to get epoxy into hairline cracks. Components with axial load (e.g., columns) typically have lower recovered stiffness due to challenges of achieving epoxy penetration in cracks closed by axial compression loads. Marder et al. (2020a) indicates that the stiffness recovery is not related to the prior inelastic drift demands, hence there is no need to relate the stiffness of a effectively repaired components to the prior ductility demand as suggested above for damaged (unrepaired) components. Instead:

- For epoxy-repaired beams the stiffness can be estimated as $K_{beam\ repair} = 0.8K_y$.
- Data regarding the stiffness of repaired columns is less reliable. It is therefore recommended that in locations of epoxy repair for columns, the stiffness recovery be ignored and the stiffness of repaired columns be taken the same as that suggested in Equation 1 for damaged components.

Damping for serviceability analyses

It may be logical to conclude that the damping in a damaged structure would be higher than the damping in an undamaged structure (Gulkan and Sozen 1974). On the other hand, damping for serviceability-level analyses is typically assumed to be less than damping for a large earthquake demand causing nonlinear response (e.g. PEER 2017). It is unclear if this reduction still applies for damaged buildings. The level of damping to be used in the linear analyses as part of the serviceability-assessment phase has not been systematically studied, but considering these two counteracting effects, it is provisionally recommended to use 5% damping as is typical for linear analysis of undamaged buildings.

Drift Limit for Non-structural Components

A key parameter in the serviceability assessment is the limiting drift capacity of critical drift-sensitive non-structural components, δ_{NS}. This is anticipated to be either cladding or partition walls. It is noted that the sensitivity of partition walls to story drift is highly dependent on boundary conditions. FEMA P-58 (Applied Technology Council 2019) provides a wealth of fragility data which may be used to select an appropriate value of δ_{NS}. Median values from appropriate fragility curves may be used, which for partition walls indicates the first damage state to occur between 0.5% to 1% drift. In the absence of detailed investigations of the non-structural components used in the building and review of appropriate P-58 fragilities, δ_{NS} = 0.5% may be assumed based on the serviceability drift limit specified in the TBI guidelines (PEER 2017).

NEXT STEPS

As noted previously, ATC-145 is an ongoing project. The framework outlined in this paper is in a state of ongoing development, and subject to further change. Current ongoing work that will drive this change is focussed on a variety of areas, including:

- Two earthquake damaged buildings are being used as case studies to validate the assessment framework and provide recommendations for improvement.
- Further analytical studies of ductile and non-ductile moment frames to better define repair triggers.
- Assessment and repair guidelines are being developed based on the technical content developed during the first two years of the project. The guidelines will accommodate additional structural systems as the framework is developed.
- Development of an assessment framework for non-ductile concrete moment frames
- Development of an assessment framework for reinforced concrete shear walls
- Development of a component repair database

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ATC-145 is an ongoing project, and conclusions reached to date are subject to change. The authors are solely responsible for the accuracy of statements or interpretations contained in this publication. No warranty is offered with regard to the results, findings and recommendations contained herein, either by the Federal Emergency Management Agency, the Applied Technology Council, its directors, members or employees. These organizations and individuals do not assume any legal liability or responsibility for the accuracy, completeness, or usefulness of any of the information, product or processes included in this publication.

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