

# Performance based assessment and design policy recommendations

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**ABSTRACT:** Holmes Consulting Group has undertaken a series of non-linear time history analyses of existing properties, including a number of essential facilities. These have been to provide specialist advice about the seismic risk, and associated recommended strengthening schemes.

These analyses have raised a number of issues which need to be fully resolved before we, as a profession, can achieve true Performance Based Assessment and Design.

What are the appropriate levels for strengthening existing buildings? Should we be strengthening to “as close as is reasonably practical to that of a new building”? Does this automatically set a *de facto* level of load to evaluate to? In turn, how do the FEMA performance limits (IO/LS/CP) directly apply to NZ codes? Recommendations based on our experiences are presented.

There are also several limitations in NZS3101 that inhibit true Performance Based Design being used to its optimal potential as a tool for the strengthening of existing buildings. There are set limits on the tie spacing in columns, regardless of the building drift and column axial load ratio. Similarly there are issues of beam column joint performance, where the existing joint steel does not comply with current codes. Recommendations for an approach to these issues are presented.

The need for clear policy and a consistent approach in developing effective strengthening schemes using Performance Based Design is highlighted.

This paper also offers some guidance as to where future research could be targeted to clarify some of the issues raised with respect to existing buildings.

## 1 INTRODUCTION

Holmes Consulting Group has developed unique in-house analysis techniques, based on the ANSR-II computer program (Mondkar and Powell, 1979), for the analysis of existing buildings. Our non-linear analysis is compliant with the most up-to-date international research and code writing, and has been adapted to incorporate new developments in materials and techniques. Holmes Consulting Group has extensive experience with seismic assessment and retrofit of existing buildings, especially historic and essential facilities.

Holmes Consulting Group has undertaken numerous non-linear time history analyses (NLTHA) of existing buildings throughout the country to provide specialist advice about the seismic (earthquake) risk associated with the buildings. A number of these have been existing healthcare facilities designated, or desired to be designated, essential (post disaster) facilities.

In performing these analyses a number of issues have been raised that restrict the utilization of the benefits of the available technology in undertaking an optimal performance based assessment and analysis.

The following sections outline some of the issues related to performance based design and assessment and especially their relevance to Importance Level 4 type structures. Recommended strengthening levels are proposed for various importance levels.

Several aspects of the development of strengthening schemes and assessments using a performance based design approach are discussed. A classification of deficiencies to be used in the assessment of existing structures is proposed and examples of this classification procedure are given to demonstrate the potential cost and time savings to the client by employing a performance based evaluation of the structure.

## 2 IMPORTANCE LEVELS, RISK CONSIDERATIONS AND STRENGTHENING LEVELS

### 2.1 Importance Levels

The recently introduced New Zealand Loadings code, AS/NZS 1170, (Standards New Zealand. 2002-2005) defines a distinction between the differences in load levels and risk factors for new buildings by the use of Importance Levels. Table 1 below summarises this.

**Table 1. Importance Levels\***

Importance Level	Earthquake Annual Probability of Exceedance	Comment	Examples
2 (IL2)	1/500	“Normal” structures and structures not in other importance levels	Hotels, Offices, apartments
3 (IL3)	1/1000	Structures that may contain people in crowds or content of high values to the community	Emergency medical facilities not designated as post-disaster, healthcare facilities
4 (IL4)	1/2500	Structures with special post disaster functions	Designated emergency facilities, medical emergency facilities with post disaster functions

\* From Table 3.2, AS/NZS 1170.0:2002

In the case of existing buildings, society accepts that there may be a greater risk<sup>1</sup>. This is partly in recognition that replacement or upgrading of the existing building to meet full code loads is unpractical and economically unachievable, and partly because there is historic, cultural and economic value in these buildings that would otherwise be lost. However, the extent of this acceptance of greater risk is dependant on function – we can tolerate a higher level of risk in buildings that do not have significant value or critical functions for the public after a severe earthquake.

To determine acceptable levels of risk and establish an effective design life for an existing building, it should be noted that an existing building’s life is dependant on many factors. These include maintenance and function – a building that has a normal design life for one use, may only have a limited life in another more intensive or critical use.

### 2.2 Legal background and requirements

The Building Act 2004 contains specific legal requirements for various situations relevant to existing

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<sup>1</sup> NZSEE White Book, DBH, Building Act 2004 policy

buildings (from a strictly structural perspective);

Section 112: Alterations to existing buildings; The building must “*continue to comply with other provisions of the building code to at least the same extent as before the alteration.*”

Section 114-115: Change of use of buildings; The building must comply with the structural code requirements “*as nearly as is reasonably practicable and to the same extent as if it were a new building.*”

Section 121-131: Buildings which are deemed to be earthquake prone (EPB); A building (of any form of construction) is deemed earthquake prone if it is likely to “*have its ultimate capacity exceeded in a moderate earthquake*” and “*would be likely to collapse causing injury or death*”.

Guidance is not specifically provided by the Act as to the necessary levels of strengthening in all situations. In the absence of this, guidelines have been developed within the industry by the NZSEE (NZSEE 2006) that have generally been accepted as reasonable. However, these guidelines do not differentiate between the different importance levels. There appears to be no legal obligations to strengthen a building above the 34% new building strength (NBS), unless undergoing a change of use, but some Territorial Authorities (TA’s) and the NZSEE guidelines recommend strengthening to a minimum of 67% NBS. For a change of use, the level of strengthening is a little more ambiguous, but generally ranges from 67% to 100% NBS depending on the local TA’s interpretation of “as nearly as is reasonably practicable” and can differ from building to building depending on what is deemed “reasonable” for each building.

### 2.3 Risk Considerations and Recommended Strengthening Levels

With a recommended level of performance we can calculate an adjusted risk factor which can in turn be used to derive the return period of the damaging earthquake. Finally, assuming a normal new building projected life of 50 years, we can determine the probability of exceedance for the damaging earthquake over the life of the building and associated risk factor, from Poisson’s formula:

$$P = \left( 1 - \left( 1 - \frac{1}{T_s} \right)^T \right) \quad (1)$$

where:  $T_s$  = the return period of the earthquake and  $T$  = design life (50 years)

In assessing recommended levels of performance, the minimum level may be acceptable for non-critical (Importance Level 2 or 3) buildings. This generally means that for Earthquake Prone buildings, strengthening to 34% of NBS can be considered. In some cases higher strengthening levels should be considered in agreement with the owner, who in turn, should be made fully aware of their statutory obligations, risk implications and future use alternatives.

There are, however, two key seismic performance objectives for an IL 4 building, the serviceability, SLS2, and ultimate limit state objectives. The SLS2 condition requires full operation after a 1 in 500 year earthquake and we recommend that this should always be achieved without compromise (this is approximately equivalent to an IL 2 event in terms of intensity). The reason for this is that structures with special post disaster functions should always be fully operational following any seismic event, to fulfil their objective.

For the ultimate limit state objective, for IL 4 structures, the minimum level of strengthening implied by the code would be 34% of NBS. This would result in a 150 year return period corresponding to a 28% probability of exceedance in a 50 year design life ie. a 14 fold increase in the seismic risk to the equivalent new building. This level of risk is believed to be too high and therefore 67% of NBS is recommended as the level of strengthening for IL 4 buildings (refer Table 2 below). It should be noted that the SLS2 requirement could be more onerous than the ULS requirement for most IL 4 buildings.

It can be seen from Table 2 that by accepting these reduced levels of performance for existing structures, there is an increase in risk of a damaging earthquake over the 50 year life of the building of between three and nine times that of an equivalent new building.

Table 2: Importance levels and implied risk levels

Importance Level	Return period of design earthquake (yrs)	Risk factor	Prob. of exceedance over 50 years	Recommended minimum performance (* not recommended) (%)	Adjusted risk factor	Return period of damaging earthquake (yrs)	Prob. of exceedance over 50 years	Risk**
4	2500	1.8	2%	34*	0.6	150	28%	14
4	2500	1.8	2%	67	1.2	800	6%	3
3	1000	1.3	5%	34	0.45	80	46%	9
2	500	1.0	10%	34	0.34	45	68%	7
1	100	0.5	40%	34	0.17	10	99%	2.5

\*\* Relative risk weighting against an equivalent new building

### 3 FEMA 356 PERFORMANCE LEVELS IN RESPECT TO NEW ZEALAND CODES

The resources required in developing evaluation procedures for all existing structural types and all materials are beyond those available in New Zealand. The most effective manner to overcome this deficit is to adopt overseas criteria where they are appropriate for New Zealand conditions. One of the most commonly used source documents is the U.S. FEMA 356 (ASCE, 2000), which has detailed criteria for a wide range of components and materials.

FEMA 356 criteria provide limits on nonlinear deformations as a function of performance levels, generally Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP). To use the FEMA procedures, we need to select a performance level which is most appropriate to the NZS 1170 ULS requirement in NZ codes. This would seem to be the CP limit, based on definitions in each document.

FEMA 356 defines the CP condition as *“little residual stiffness and strength, but load bearing walls and columns function. Building is near collapse.”*

NZS 1170 defines the ULS as *“the structure loses integrity, becomes unstable or loses equilibrium... but does still retain a small residual load bearing capacity that prevents local or global collapse.”*

Based on this, our recommended policy is to adopt FEMA 356 criteria for deformation limits of components, with the CP limits being assumed appropriate for the NZS 1170 ULS seismic loads.

In addition, we recommended the adoption of FEMA 356 Immediate Occupancy (IO) level to correspond with the NZS 1170, SLS2, operational continuity requirements for IL4 buildings.

### 4 CLASSIFICATION OF DEFICIENCIES

In terms of the Building Act 2004 legal definitions, an EPB will meet two conditions:

- a) Will have its ultimate capacity exceeded in a moderate earthquake; and
- b) Would be likely to collapse.

The NZSEE Study Group considers that the second condition is ambiguous and would prefer it be deleted. However, in practice it can be useful because codes and guidelines make it simpler to define the ultimate capacity of individual components than of complete buildings. Therefore, the second condition can be useful when the ultimate capacities of individual components are evaluated.

For new buildings, all components are designed so that they do not meet condition a) for the design

level earthquake, and the overall building will then also not meet condition b). For existing buildings, there may be some components which meet the first condition but, nevertheless, the building does not meet the second criterion and so the building is not earthquake prone.

As an example, a concrete frame may have a small number of beams which have insufficient shear reinforcement. These components have their ultimate capacity exceeded in a moderate earthquake. However, in the absence of other deficient components the building would not be likely to collapse and the building would not be earthquake prone. On the other hand, if the shear steel deficiencies are in the ground level columns then the frame will have a high likelihood of collapse and the building would be earthquake prone.

In guidelines in common use (NZSEE and FEMA) no consideration is given as to the type of failure expected in a building and to what deficiencies are considered “critical deficiencies”, ie. which structural elements will actually cause failure of the structure if loaded beyond their CP limit?

This limitation in the NZSEE and FEMA guidelines, as well as NZS3101 (Standards New Zealand, 1995 & 2006) and other material codes, restricts true performance based design and assessment, and the optimum utilisation of the benefits from available techniques, for strengthening existing buildings. This is because, in the absence of specific guidance, the usual procedure is to strengthen any elements which exceed the CP limit. However some “failures” are not critical deficiencies in a building. Remedying every deficiency, even if not critical, may penalise a building’s strengthening scheme by making it extremely costly. It may even be counterproductive, in that the immense cost of strengthening every element may deter an owner from any strengthening, thereby not remedying a smaller number of critical deficiencies.

As our company implements performance based evaluations, we are assessing the consequence of failure rather than the failure itself and developing internal classifications for deficiencies, a sample of which are listed in Table 3. Generally, our recommendations to clients are that they assign priority to deficiencies which make global collapse likely. These are generally in the vertical load supporting elements, the columns and wall piers.

**Table 3: Classification of Local Deficiencies**

<b>Component</b>	<b>Deficiency</b>	<b>Collapse Likely?</b>
Concrete Column	Excessive plastic rotation	No
	Insufficient shear steel	Yes
	Insufficient volume of confining steel	Yes
	Tie spacing too large	Yes/No (ref 4.1 below)
	Excessive axial stress	Yes
Concrete Beam	Excessive plastic rotation	No
	Insufficient shear steel	No
	Insufficient volume of confining steel	No
	Tie spacing too large	No
	Excessive axial stress	No
Beam Column Joints	Excessive shear stress	No (ref 4.2 below)
Shear Wall Pier	Excessive shear strain	Yes
	Excessive axial stress	Yes
Shear Wall Coupling Beam	Excessive shear strain	No

The classifications in Table 3 are based on engineering judgement and some are potentially contentious. We feel the profession should address this aspect of evaluation as performance based procedures become more common practice. In particular, two items need input from researchers, as discussed below.

#### 4.1 *Anti-Buckling Steel*

NZS3101 specifies that the spacing of the confining steel shall not exceed the lesser of one-third the least lateral dimension or 10 diameters of the longitudinal bar being restrained. In potential plastic hinge regions this reduces to one-quarter of the least dimension or 6 bar diameters. This requirement was introduced in the 1995 code and is more stringent than in previous codes.

Strict compliance with this would mean that any building with concrete columns, built before about 1995, is likely to require almost all gravity columns to be strengthened regardless of the building drift and axial load ratio. If the column element has a low axial load ratio and a small drift, is buckling likely to occur? The tie spacing requirement is intended to restrain flexural reinforcement from buckling after cover concrete has spalled off (NZS3101 C8.4.7). Provided the ductility demand is such that cover concrete does not spall then spacings exceeding the NZS3101 limit will not form a critical deficiency. Currently, we use a limiting concrete strain of 0.004 to ensure spalling does not occur. A longitudinal reinforcement strain limit of 0.005 has been suggested elsewhere. Guidance is required as to what limit should be placed on concrete and/or steel strain to prevent buckling of main reinforcement. Research in this area is underway at the University of Canterbury and it is hoped it will provide practical guidelines to practitioners.

#### 4.2 *Beam-Column Joints*

Beam-column joint deficiencies usually occur in exterior and corner columns with earthquake imposed axial loads. The joint concrete shear strength is a function of axial load and joints are usually deficient under minimum (tensile) load and not under maximum (compressive) load. Joint deficiencies are very difficult to remedy, especially in two way frames.

There are a number of areas where guidance is needed. Do these joints form a collapse hazard, given the deficiency only occurs when the load is upward? Can joint deficiencies be assessed in analysis by limiting the strength of framing beams to the moment capacity that causes the joint to reach its strength? Using the NZSEE guidelines, the concrete shear strength factor,  $k$ , is 1.0 for interior joints and 0.4 for exterior joints. Can the exterior value be increased if the joint has beams framing in from the side? Research in this area is required and recommended.

### 5 EXAMPLES OF PERFORMANCE BASED ASSESSMENTS

#### 5.1 *Example 1*

An example of the application of these classifications of deficiencies is as shown in the health care facility in Figure 1, which is an Importance Level 4 structure. The roof and floors have been removed from the model view for clarity. The recommended strengthening level is 67% of new building strength, equivalent to an  $R = 1.2$  level of load. At this level of load, there are numerous beam and column shear, column plastic rotation, confining steel volume and spacing deficiencies as defined in FEMA 356 and NZS3101, refer Figure 1.

In terms of codes and guidelines, there are a significant number of elements that would require strengthening to meet the criteria that all components are within their ultimate limit state. However, of these, only three column shear deficiencies and four column confining steel volume deficiencies would pose a likely risk of collapse (refer Table 3). From the discussion in section 4.1 it can be concluded that since the drifts of the building are so low and the axial load ratio small, that failure due to insufficient tie spacing is not likely. The recommended action is therefore to apply remedial measures only to the seven ground floor columns shown red in Figure 1. The elements that fail code and guideline limits but do not pose a collapse risk have been coloured green. It can be seen that vast cost and time could be saved if the strengthening was only performed on the seven ground floor columns posing a collapse risk to the structure.

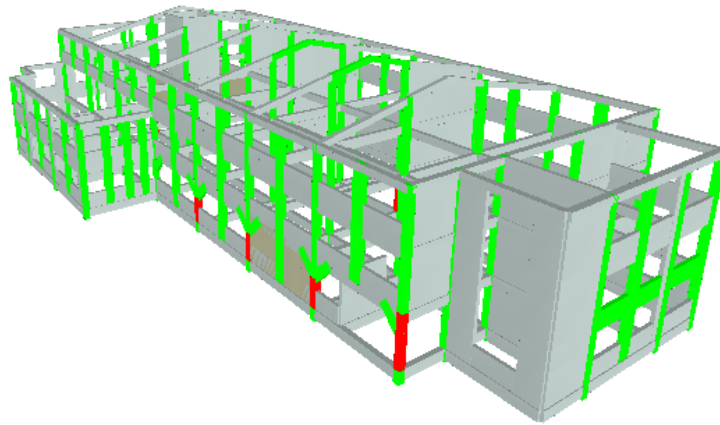


Figure 1. An Importance Level 4 Building showing damage at R=1.2 load level.

### 5.2 *Example 2*

The healthcare structure shown in Figure 2 below was originally analysed using traditional techniques and an extensive strengthening scheme was proposed. When analysed using more accurate NLTHA performance based techniques by Holmes Consulting Group the strengthening of the building was able to be almost completely eliminated.

Analyses by Holmes Consulting Group showed a small number of beams with shear deficiencies due to plastic rotations in the beams, see Figure 2, and several beam column joints with insufficient shear strength under tensile loads. Using the critical deficiency table above, these shear deficiencies are deemed non critical and would not pose a collapse hazard to the structure, although the beams are likely to suffer some local damage. Hence, essentially no strengthening was proposed for this healthcare structure to meet the recommended load level.

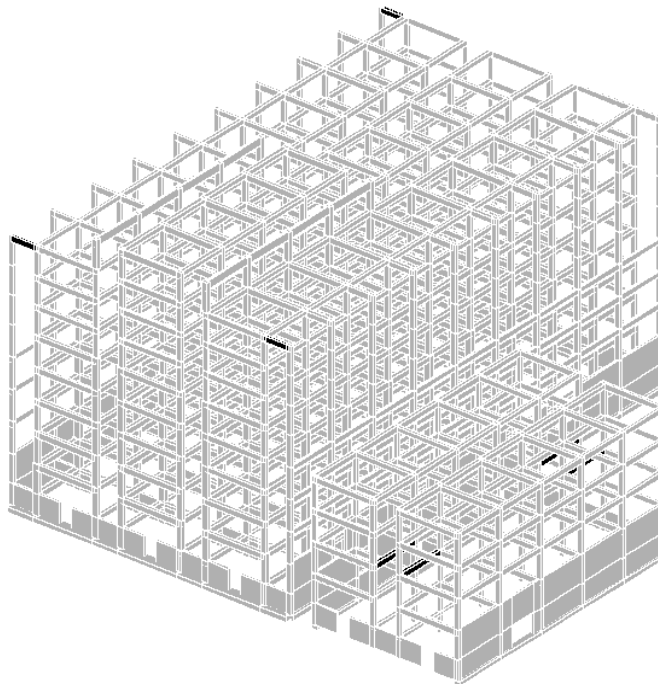


Figure 2. A healthcare structure showing potential beam shear failures at the recommended load level.

## 6 SUMMARY

Holmes Consulting Group has undertaken a significant number of non-linear time history analyses to provide specialist advice about the seismic risk, and associated strengthening schemes for a number of the existing healthcare facilities deemed to be essential (IL4) facilities.

Strengthening levels are recommended for various Importance Level structures. Table 2 shows that by accepting a reduced level of performance for existing structures, there is an increase in risk of a damaging earthquake over the 50 year life of the building of between three and nine times that of an equivalent new building. For Importance Level 4 structures an increased minimum level of strengthening to 67% is recommended, provided that the SLS2 criteria are met in full. The increased risk of a damaging earthquake when only strengthening to 34% NBS, of 14 times that of an equivalent new building, is deemed excessively high.

The NZSEE and FEMA guidelines do not address the consequences of the type of failure. In the absence of specific guidelines in this area, the usual procedure is to strengthen any elements which exceed their CP limit. Applying strengthening to every deficiency, even if not deemed to be a critical failure, may penalise a buildings strengthening scheme by making it extremely costly. A classification of critical deficiencies is proposed (refer Table 3) based on engineering judgement. Generally, our recommendations to clients are that they assign priority to deficiencies which make global collapse likely. These are generally in the vertical load supporting elements, the columns and wall piers.

Examples of the application of these classifications is shown for two essential facilities where more accurate NLTHA performance based techniques have been used to reduce the need for costly, time consuming strengthening where non-critical deficiencies do not pose a collapse hazard to the structure.

Two specifically identified deficiencies that require specific guidance and debate are presented. Firstly, the requirement in NZS3101 in regard to the spacing of the confining steel in concrete columns. If the building has a small drift and the column element has a low axial load ratio the likelihood of bar buckling is significantly reduced. This is not reflected in the code. Guidance is required as to what limit should be placed on concrete and/or steel strain to prevent buckling of main reinforcement.

Secondly, the NZSEE sets the concrete shear capacity of exterior joints at 40% of that of interior joints leading to many joints “failing” an analysis. Guidance is required by researchers as to the collapse hazard of these “failing” beam-column joints and also whether the capacity can be increased if transverse beams frame into the joint.

This paper identifies a number of issues which need to be fully resolved before we, as a profession, can achieve optimal Performance Based Assessment and Design. In addition, the need for a clear policy and a consistent approach in developing effective strengthening schemes using Performance Based Design has been highlighted.

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