WHICH STRUCTURAL SYSTEM IS BEST?

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ABSTRACT: *This paper discusses some tools that may assist designers to determine the building system which is "best". The choice of building structural system may be treated as a constrained optimization problem considering uncertainty. Real life constraints affecting the design include owner requirements (e.g. functionality, aesthetics), political and safety considerations (such as height limits, performance, waste disposal criteria), cultural/logistic considerations (e.g. skill sets for construction, material availability), as well as cost. These decisions may be made in a number of ways. Decision support tools using both probabilistic methods (for buildings in seismic and non-seismic design zones), as well as using subjective quantitative analysis, are described. Examples of both are provided for the selection of different building systems. It is shown that both probabilistic design tools and subjective quantitative analysis tools have strengths and weaknesses and often they are used together. Since both methods are based on many assumptions, interpretation of the outputs from such tools should acknowledge such assumptions.*

KEYWORDS: Subjective quantitative assessment, probabilistic methods, decision making, structural design, earthquake engineering

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1 INTRODUCTION

The job of a structural engineer is to provide a structure for their client. Clients generally believe that they are paying engineers to produce the *best* structural system, but it is not always clear what the word "best" implies.

Presumably the best solution satisfies design objectives; both those known at the time of design, and those that develop during the life of the structure. Because different stakeholders are likely to have different objectives, the "best" is not necessarily the same for all people. Furthermore, because the future is unknown at the time of design, the true best solution cannot be known. The word "best" is used below to indicate the structure that will satisfy all the expected constraints known at the time of design of stakeholders interested in ensuring the performance of the building over its design life considering uncertainties.

The best system is *not* generally that with the best performance, as it is possible to keep improving the performance usually with more expensive systems. The best system is always constrained by cost. This is emphasized by the following quote, attributed to a founding member of the Institution of Civil Engineers who stated that "*A Civil Engineer can do for a penny what any fool can do for a pound*" (Bearman, 2014 [1]). Structural engineering has long been recognized as a constrained optimization problem given uncertainty [2]. The problem is solved differently with different constraints. Some examples of methods to solve this problem with the following constraints are:

a) Minimum Initial Cost

This method is commonly used throughout the world in all countries. Generally the client and/or engineer makes the assumption that current standards provide sufficient safety for good performance and it is the engineers job to satisfy the standards in the most economical way possible. Legal systems around the world are such that a designer who satisfies the relevant standards is not likely to be liable in the event of collapse or damage. A designer who provides a better, or more robust design, may be discriminated against because of cost. The first author knows of one US company which uses a number of standards and determines the minimum member sizes from the most liberal of the standards. Such an approach is advantageous to developers whose aim is to build and quickly sell their buildings.

b) Minimum Life Cycle Cost

For clients who not only construct, but also need to use and maintain their structures, whole of life performance is often considered. It is often used for bridges which are owned by the state. A number of different techniques can be used to assess the life cycle performance. These include life cycle assessment (LCA) tools which may include environmental, maintenance, and other factors.

While the lowest initial cost may be the optimal choice for the developer, other considerations may be optimal for the citizens of a city, and for the environment. For example, (i) in Germany many houses are required to be designed to be very strong and require low maintenance as this has a lower longer term overall cost to the society; (ii) for a low-damage structure to be usable immediately after an earthquake, the neighbouring structures should be designed such that they do not become damaged and threaten nearby buildings (e.g. Hotel Grand Chancellor [3]). Authors such as Goda and Hong [4] propose that factors such as risk attitude, societal tolerable risk level, and life quality should be considered in making the optimal decision.

In this paper, it is assumed that current standards provide a minimal level of safety, but more severe constraints may result in a better structure. Emphasis is on the type of construction that will provide the greatest benefit over the building life, especially considering earthquake effects. Two decision support tools to assist in determining the most appropriate building type are compared – one using *probabilistic techniques*, and the other using *subjective quantitative assessment* (SQA). In particular, answers are sought to the following questions:

a) What are the benefits of probabilistic methods?
b) What are the benefits of SQA?
c) What are some current problems to which these methods can be applied?

2 PROBABILISTIC METHODS

2.1 NON-EARTHQUAKE DESIGN

For non-earthquake design Working Stress Design, WSD (also called Allowable Stress Design, ASD) [5] has dominated in the past. It did not have any formal probabilistic basis and the factors of safety resulting from experience seemed to work quite well. In WSD the safety factor is included by reducing the strength of the materials to something significantly less than the actual nominal level, while using expected loads.

In the 1980s, a new methodology based on probabilistic concepts termed Limit State Design (LSD) was developed. This is also referred to as

Load Resistance Factor Design (LRFD) [6]. This was promoted as being superior to WSD for a number of reasons (Salmon and Johnson, 1995 [7]). These reasons include (a) safety is provided appropriately with the parameter considering the uncertainty, (b) it is rational enabling better decisions, (c) it provides more consistency for different load cases and materials as all structural elements could be calibrated to provide a uniform probability of safety against collapse, (d) it enables new materials to be used more easily in design, (e) it allows structures of different materials to be considered using the same approach making it easier to design a structure with different materials, and (f) second-order effects, which are related to the demand, can be considered directly.

Despite its advantages, recommending design load values, γ_i, and resistance factors, ϕ, for LSD has had its difficulties. While it may be possible to provide all structural elements with the same probability of failure, P_f, it is necessary to select the P_f to be used. If P_f is too low, then the structure may be uneconomical. If P_f is too high, then the structure may be unsafe. However, the determination of P_f is as much a political as it is a technical consideration. Ellingwood et al. (1982) [8] decided that for LSD it was best to select a value of P_f which would result in similar sized members to that resulting from WSD. This could then be modified as knowledge improved. It is interesting to note that the P_f used for horizontal loading (e.g. wind) is greater from that used for vertical loading (e.g. gravity and snow loads).

So while many advantages of LSD may exist, because of the calibration of the method, the end result is essentially no different from that of WSD. This is shown also by the fact that the latest US AISC code for the design of steel structures allows either WSD or LSD to be used [38].

2.2 EARTHQUAKE DESIGN

In earthquake resisting design, there has been an interest in probabilistic methods. However, before these became popular, we used *Performance Focussed Design* (PFD) or *Performance Focussed Seismic Design* (PFSD). Some people have referred to this using the terminology *Performance based design* however this terminology is easily confused with true *performance based codes* such as those of Hammurabi [9] which depend on the *actual performance* of that of the constructed structure.

None of our codes/standards are related to the *actual structural performance*, but they are prescriptive (with differing degrees of prescription). However they have a *performance focus* and should therefore be termed *performance focussed* documents.

Standards incorporating PFD have been around for many years and they have not had a rigorous probabilistic basis. Paulay in the 1980s used to teach classes stating that three levels of performance under various levels of ground shaking were considered in earthquake codes. The performance requirements were (i) No non-structural damage in small earthquake shaking, (ii) No structural damage in moderate earthquake shaking, and (iii) No collapse or life loss in strong shaking. These multiple performance objectives were met by respectively providing (i) stiffness according to drift limits, (ii) strength according to the design forces, and (iii) system ductility through detailing and capacity design considerations.

Since the 1990s the term *Performance-Based Design* has been used (as though it were new) for earthquake-resistant design [10]. This generally refers to *multiobjective probabilistic performance focussed design* (MPPFD) which is more explicit than the older PFD about each limit state considered. It also uses probabilistic methods to describe a likelihood of these performance objectives being met. The more advanced/complex analysis techniques (which may also be used in traditional code design) such as nonlinear response history analyses (NRHA) may be used in order to assess the response and the chance or reaching a particular limit state. This can be extended to estimate loss due to earthquake.

MPPFD (commonly called PBD) is generally used as an *alternative solution* [39] for larger or irregular/unusual buildings in order to obtain more economical member sizes that those using standard methods. However, the economic advantage is not as apparent for NZ codes as it is for other codes such as those in the US. This is because the simplest methods, such as the *equivalent static procedure*, has already been calibrated to estimate likely actual response so that more advanced analysis techniques often offer little advantage. This is due to the incorporation of higher mode effect factors thereby reducing the expected demand in the NZ code. As LSD is to WSD, MPPFD also has some similarities to the older PFD approaches.

Probabilistic methods can be applied to evaluate likely losses. Often the PEER equation, Equation 1, is referenced [11, 12]. It can be used to develop either *scenario loss* (for a particular event), or *probabilistic loss* (over a certain time). Here, *IM* is the intensity measure of shaking, *EDP* is the engineering demand parameter, *DM* is the damage measure, and *DV* is the decision variable. This DV is usually in dollars, caused by the 3Ds - damage, death (and injury), and downtime.

$$P(DV) = \iiint P(DV \mid DM) P(DM \mid EDP) P(EDP \mid IM) P(IM) dDM.dEDP.dIM \quad (1)$$

Loss assessment techniques allow many parameters to be included in assessing the total loss. The loss can be presented in many ways [12] including in a breakeven analysis (MacRae [13]), to determine what structure has the lowest losses.

Loss assessments consider some aspects of aleatory uncertainty (which can be described by a statistical variation about an estimate of the behaviour) and epistemic uncertainty (which is the error or bias in the simple model used to describe real behaviour) and tries to do this in a rigorous way. Ontological uncertainty (which is due to things which are not even considered) acknowledges that the models used may not consider some of the most important parameters. Some authors (e.g. Taleb [14]) indicate it is the highly improbable events that are most significant. In general, the more complex the methods become, additional uncertainty is introduced compromising the final results in terms of loss.

Bridge management systems (BMS) are another field where Life Cycle Cost analysis can be comprehensively used and elegant methods have been advocated especially by Frangopol and his group including things like the effect of genetic algorithms [e.g. 15]. The more sophisticated of these methods rely on significant data to calibrate them properly. Because of the amount of data required, and the difficulty of obtaining and processing high quality data, bridge management organisations generally use relatively simple techniques (E.g. Hanshin Expressway Company).

2.3 PROBABILISTIC EXAMPLE

A loss assessment considering only direct repair costs of two 10 storey concrete buildings was considered (Yeow et al., 2013 [16, 17]). One building contained walls to resist the lateral loading, while other was a moment frame building. It was suspected that the wall building would behave best based on Fintel (see Murthy [18]) and the relatively good behaviour of buildings in the last Chilean earthquake (EERI 2010 [19]).

Response history analysis (RHA) can be used to find the variation in response demand (EDP) at different levels of input excitation (IM). This was then combined with other relationships in Equation 1, as well as the likelihood of IM from a hazard curve, to estimate the loss.

One example of loss is that from the components given in Figure 1 where it may be seen that the drywall damage in both frame and wall structures had the greatest percentage loss in terms of component cost during design level events. The proportion of damage costs of each component in relation to the total damage cost of the structure can also be identified. This information can assist with decisions as to what steps can best be made to reduce total earthquake loss.

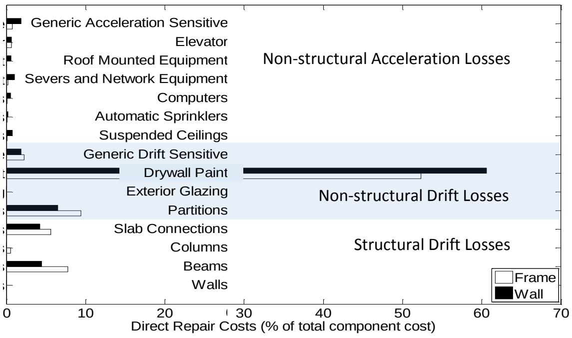


Figure 1: Mean loss in a 10% in 50 year event deaggregated by components [16]

Breakeven analysis [13] can be used to consider the difference in costs with time. Here, the cost of the frame structure was \$12.91million. As seen in Figure 2, the initial cost of the wall structure was \$120,000 more than this. While the frame structure has a higher rate of loss than that of the wall, the total losses are always less than the wall losses. For other types of comparison, or inclusion of other types of loss such as injury or downtime, the difference in loss may be such that the total loss curves cross over each other at some point. This is the break-even point. The curve with the lowest total loss at the design life of the structure is therefore the best choice. In this case, the frame structure always has lower total losses because of its lower initial cost. The non-linearity in this graph is due to the discount rate.

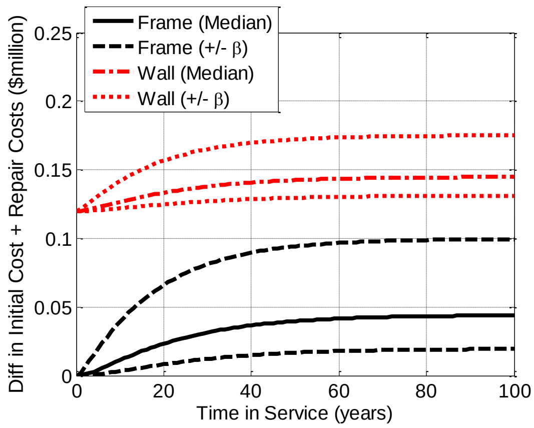


Figure 2: Break-even analysis of different options [16]

Another way to compare structures is to use graphs showing the annual rate of exceedance of different total costs/losses (including the effect of initial cost) as shown in Figure 3. Here for example, the expected total cost of an event with an annual rate of exceedance of 0.002 (i.e. a 500 year earthquake) is similar for both the frame and the wall. For more frequent events, the frame has the lowest total costs, but for larger events it is the wall. Techniques to consider the total loss over its lifetime have also been described (Yeow et al. [2014])

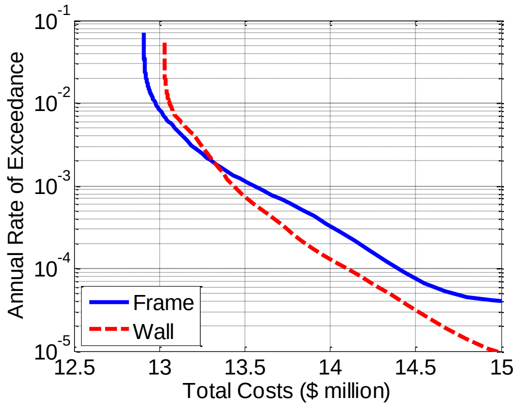


Figure 31: Comparison of loss hazard [17]

2.4 PROBABILISTIC LIMITATION

While the probabilistic methods above provide useful information, they are resource hungry requiring data and relationship information:

i. about the faults in a region affecting the structure being analysed
ii. about the frequency of rupture of those faults
iii. about the effect they will have on a site
iv. about the effect of soils and the structure on the structure base excitation
v. to obtain an appropriately large suite of reasonable ground motions
vi. about the structural response given the base excitation (which depends on the modelling and the analysis methods)
vii. about the likely damage and loss given the response

The data should not cost too much and it must be of high quality and relationships should be calibrated. To get good data is no easy task. For particular buildings the building materials, structural form, soil conditions, fragilities etc. must be known. For example, efforts to improve the data for the NZ regional loss estimation software RISKSCAPE, started in 2004, are likely to continue for many years from now. Data is being gathered through various means, including via meetings with engineers to discuss the damage observed in specific buildings during the 2010-2011 events.

Also, the process of estimating losses is complex. It requires convolution integrals, and the use of the total probability theorem, creating a sense of mysticism to those who are not familiar with these techniques. It may also be difficult to find errors in the analysis, so checks are required at all stages of the process if the results are to be seen to be reasonable.

All influential effects need to be considered if there is to be any confidence in the result. For example, after the 2010-2011 Canterbury earthquakes, EQC stated that losses due to ground movement and aftershocks were not properly considered and one half of all losses were associated with liquefaction. While these effects were known about, efforts to develop techniques to predict these effects and appropriately incorporate them into loss models had not been done by the major insurers around the world.

Concerns have been raised about the state of modelling in NZ by Carr [21]. Even it if is conducted without error, different assumptions can result in very different estimates of response. This has been illustrated by Hopkins in his earthquake response *uncertaintree* illustration [40]. Here different assumptions about each of the many different aspects of the model can cause divergence and different answers.

It can be seen from the above discussion that there are a number of limitations in probabilistic loss assessment. Even though some analyses, such as those by Yeow et al. [16, 17] (which were carefully undertaken), contain many assumptions. It is also not easy for readers to know whether the values for the mean loss (for example) are reasonable. For this reason, many researchers regard these tools as being useful to compare the behaviour of different types of structure, without being numerically accurate. Of course, the insurance industry is concerned about the actual magnitude of the damage and must weigh up the need for complexity, with its increased consideration of different effects, with the resulting increase in uncertainty in better estimating likely losses.

Simpler approximate methods of conducting the probabilistic loss estimation have been proposed and match well with more complex analyses for some cases [E.g. 34, 35]. However, while these simplify the process to get results, many of the major issues regarding input data accuracy remain.

The discussion above, while describing some of the uncertainties of probabilistic methods, is not to negate their usefulness. Estimates are needed to compare building loss and to evaluate losses for economic and political reasons, and probabilistic methods offer much in this respect. However, the output is only as good as the input provided.

Probabilistic methods often include SQA within them. For example, with hazard analysis different weightings may be subjectively given to different methods in a logic tree approach [36].

3 SUBJECTIVE QUANTITATIVE ASSESSMENT (SQA)

3.1 SQA METHODOLOGY

SQA is used in many fields including structural engineering. It does not rely on advanced mathematics (although it may use some information that does).

SQA involves rating different characteristics in terms of what may be better according to a subjective scale. Some examples of its use are:

a) to assess the best presentation for an award. Points between 0-3 say, are given for the (1) content of the presentation, (2) manorisms/style of the presenter, (3) quality of visual aids used, and (4) ability of the presenter to answer questions. These points for each characteristic category is added up (e.g. 0-12) allowing a simple comparison with other presenters. The process is very simple. It contains a degree of subjectivity (as the numbers specified in each category are based on impressions which are related to experience/biases, etc.) as well as a level or arbitrariness (for example – here all categories have the same number of points and same weighting). The weightings of a number of assessors may be combined together to obtain an overall assessment as well.

b) to decide what brand of car to purchase, considering characteristics such as cost, reliability, economy as well as other issues which appeal to the owner such as name brand, colour etc. Factors such as colour may relate to preference, perceived safety, or to other issues – E.g. a blue car tends to attract bumblebees more than cars of other colours, so may not be preferred by someone with a bumblebee allergy. While a specific vehicle may have the best benefit considering cost, other factors such as availability of funds may mean that this is not judged to be the best decision.

Further applications of SQA include selection of a profession, house, hobby, investments, as well as any other situation where it is necessary to select between 2 or more options.

The SQA is easiest to perform when all of the data is known, so that a full comparison may be made. However, in general, all of the information required to make the decision is not available, and there is considerable uncertainty. This can also be factored into the assessment. When decisions involve other people (e.g. in the case of the Prisoners Dilemma [23], at an auction, in a game of poker, or in intergovernmental politics, such as the case illustrated by the Cuban Missile Crisis [24]), factors such as trust play a greater role in the decision. These are beyond the scope of this paper.

3.2 SQA EXAMPLE

Chanchi et al. (2012) [25] have recently applied SQA to different steel systems to quantify them in terms of their damagability. Those with low damage are judged to be the best (MacRae and Clifton [26], MacRae [27]). They include traditional systems which involve significant yielding such as Moment Resisting Frames (MRF), Eccentrically Braced Frames (EBF), Concentrically Braced Frames (CBF), Buckling Resistant Braces (BRB) and steel Plate Shear Walls (PSW) as shown in Figure 4. Improved systems including Reduced Beam Sections (RBS), Eccentrically Braced Frames with Replaceable Links (EBFRL), Post-Tensioned Steel Frames (PTSF), and Rocking Frames (RF) are considered in Figure 5. Low damage frames with High Force to Volume (HF2V) lead dissipators, with Sliding Hinge Joint (SHJ) friction connections, and with braces with Asymmetric Friction Connections (AFC) or Symmetric Friction Connections (SFC) are described in Figure 6. More detail and references to such systems is given in [25, 26]. Many other systems and variations exist, but they have not been included in this simple study.

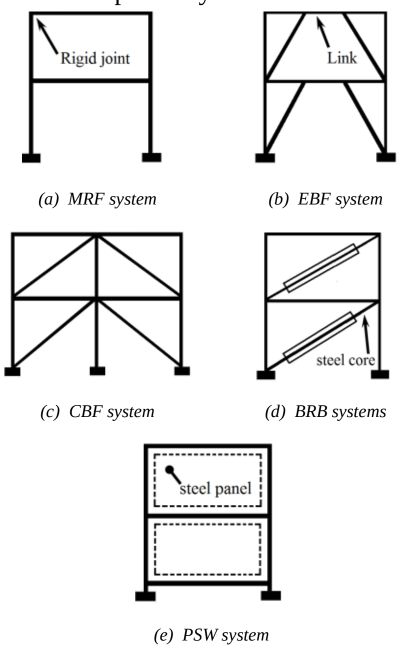
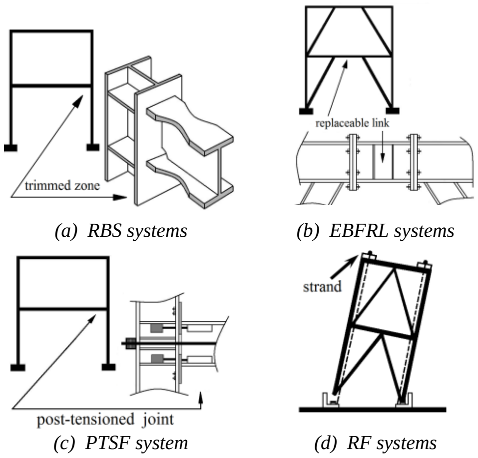


Figure 4: *Traditional steel systems [25]*

The seismic sustainability and damageability are defined in terms of the following damage indicators: Drift Damage [DD], Element Replaceability [ER], Floor Damage [FD], and Permanent Displacement [PD]. Integer ratings are given for each damage indicator.

The Drift Damage [DD] is related to the structural system including the gravity frame. Values up to 3 are given for systems considered to have large displacement demands, or those which were considered to need replacement after a low number of earthquakes.



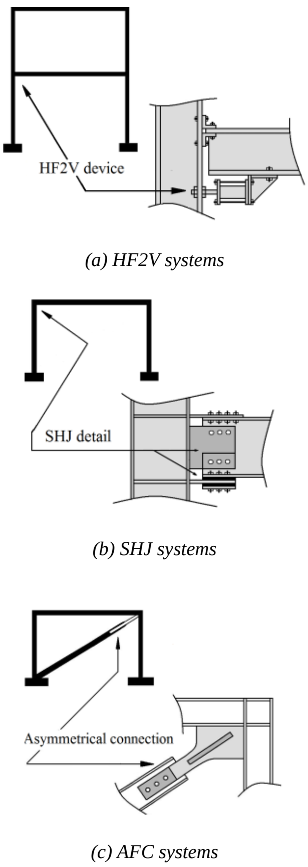
(a) RBS systems (b) EBFRL systems
(c) PTSF system (d) RF systems

Figure 5: Improved joint or element steel systems [25]

For systems where it is difficult to replace the damaged elements, Element Replaceability [ER], scores of 2 may be given. The ER rating is lower for elements not in the primary gravity resisting system (such as braces) rather than those in the system (such as beams in MRFs, or columns).

The relative likely permanent (or residual) displacement [PD] of a structure after an earthquake is indicated by the hysteresis loop shape (MacRae 1994 [28]). Fatter loops, such as the elasto-plastic loop in Figure 7a, generally are associated with greater permanent displacements than those with positive post-elastic stiffness such as that shown in Figure 7b. Furthermore, loop which are pinched to the extent that they have zero displacement at zero, such as that shown in Figure 7c, have no residual displacement. Systems with hysteresis loops associated with large permanent displacements are likely to be given a rating of [2], while those with no permanent displacements are given a [0] rating.

Floor damage is not desirable in a low damage structure, so it should be avoided. It can be significant in eccentrically braced frames with links in the beam which undergo large inelastic strains [42]. Also, in post-tensioned beam systems (PTSF) gapping occurs at the end of the beams during large deformations as shown in Figure 8a [26, 29]. This pushes columns apart causing additional demands particularly on the columns as shown in Figure 8b [30] and severe slab damage as shown in Figure 9 [31]. This damage can occur in buildings made of any material. The NZ Structural Engineering Society (SESOC) states that "floor diaphragms must be detailed to accommodate significant frame elongation" and notes that providing this detailing is no easy task [41]. In general it is not possible to provide appropriate detailing, so damage is expected in gap opening systems [26].

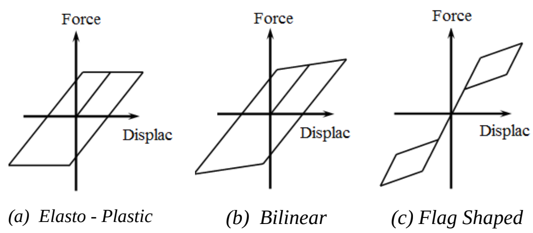


(a) HF2V systems

(b) SHJ systems

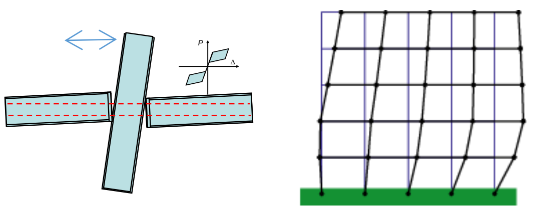
(c) AFC systems

Figure 6: Some low damage steel systems [25]



(a) Elasto - Plastic (b) Bilinear (c) Flag Shaped

Figure 7: Hysteresis Loops [25]



(a) Gapping (MacRae, 2010 [26, 29]) (b) Resulting Frame Distortions (Kim et al. 2004 [30])

Figure 8: Post-Tensioned Beam Effects

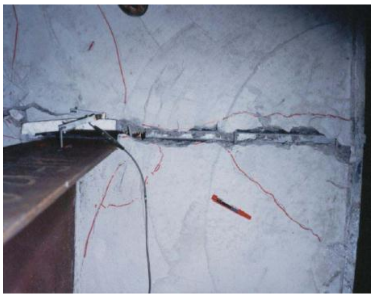


Figure 9: Slab Damage in a Post-Tensioned Beam Subassembly (Clifton, 2005 [31])

The sum of all the damage indicators above is 9 giving a value between 0 and 9, with 9 indicating a damage prone structure. The seismic sustainability was simply defined using Equation 2. A structural system with a score of 9 is has a high seismic sustainability.

$$Sustainability = 9 - \Sigma\ (damage\ indicator)_i \quad (2)$$

The SQA are given in Table 2 for the different frames which are assumed to be designed for the minimum code criteria and subjected to a design level earthquake. As expected, those with the lowest damage value (out of 9) and the highest sustainability (out of 9) are the low damage systems. Greater background is given by Chanchi et al. (2012) [25].

Limitations of this method are obvious. Firstly it is subjective not only in terms of the ratings given, but also in terms of the criteria chosen, and the weightings of each of these criteria in the final rating. The way the seismic system is designed and connected to the rest of the frame will also influence the demand. Also, this example does not explicitly consider initial cost (but this could be added as another category).

Table 2: SQA summary (Chanchi et al. 2012) [25]

CATEGORY	SUBCATEGORY	ER	FD	DD	PD	Damage	Sustainability
		[0 – 2]	[0 – 2]	[0 – 3]	[0 – 2]	[0 – 9]	[0 – 9]
Traditional Systems	Moment Resisting Frames – *MRF*	2	1	3	2	8	1
	Eccentrically Braced Frames – *EBF*	2	1	2	2	7	2
	Concentrically Braced Frames – *CBF*	2	0	2	2	6	3
	Buckling Restrained Frames – *BRB*	0	0	3	2	5	4
	Plate shear walls – *PSW*	1	1	2	1	5	4
Improved Joint or Element Systems	Reduced Beam Section – *RBS*	2	1	3	2	8	1
	EBF with replaceable link – *EBFRL*	0	1	2	1	4	5
	Post-Tensioned Steel Frames – *PTSF*	1	1	2	0	4	5
	Rocking Frames – *RF*	1	1	2	0	4	5
Low Damage Systems	Lead Extrusion Dissipators – *HF2V*	0	0	0	1	1	8
	Sliding Hinge Joint – *SHJ*	0	0	1	1	2	7
	Asymmetrical Friction Connections – *AFC*	0	0	1	1	2	7

However, even with these limitations, it has proven to be a very effective tool to communicate to stakeholders and has resulted in changed decisions. Also, this type of critical evaluation has become reinforced as part of the thinking of stakeholders considering various structural decisions.

Reasons for this are likely to be its simplicity, the ability of the stakeholder to change the weightings, or add another category. Stakeholders are empowered to use the system and make it their own. They can easily develop it, modify it, defend it, and explain it with confidence to others. They can also include information within it such as that from probabilistic analysis giving the results a weighting that is considered to be appropriate.

It should be noted that this study could also have been conducted with probabilistic methods, but this is particularly time consuming, especially at the preliminary design stage.

4 OTHER COMPARISONS

Some comparisons which are currently topical in NZ engineering include the following. Probabilistic methods and SQA can be applied to these. They include:

4.1 BASE ISOLATON SYSTEMS

Different types are available, including lead-rubber bearings, laminated rubber-steel bearings, friction pendulum systems, and many others. Each has its own range of costs and performance. Some base isolation devices have different post-elastic stiffnesses than others. Those with a high value cause greater forces on the structure resulting in an increase drifts and accelerations which could cause damage. However, a lower post-elastic stiffness could result in an increase in peak and permanent displacements. Some bearings have a lateral resistance which is related to their axial force. Therefore, in buildings where there are significant areas over the plan where columns carry more stories than others, the same isolator can be used. Other isolators need to be provided with a width according to the lateral resistance required, and small isolators may have a higher chance of running out of displacement capacity. Analytical methods with a probabilistic basis can be used to assess the likely demands of the different systems, and the likely total losses of the different systems.

4.2 SITE HAZARD LEVELS

It is often considered that seismic hazard analysis, in order to obtain the likely hazard at a site, is based on standard computational procedures. The fact is, that there is substantial subjectivity, particularly due to the lack of information available based on past shaking and attenuation relationships. In Christchurch, this was illustrated recently by the post-earthquake change in zone factor [37]. Factors affecting this decision were (i) 500 year shaking level, (ii) the expected shaking level in the lifetime of new structures, (iii) expectations of the public that design levels should rise, (iv) the desire not to increase the design shaking level greater than that in what is considered to be NZs most seismically hazardous large city (Wellington), (v) the desire to not require buildings already retrofit to 100% of the old building standard to need to be retrofit again, and (vi) the desire to decrease the risk of damage to non-structural and structural elements in smaller events. This was changed by modifying the design levels for ultimate and serviceability limit states. However, the final decision, while containing input from hazard modellers, was also made considering other factors. This involved probabilistic and SQA methodologies.

4.3 EARTHQUAKE RECORD SELECTION

In probabilistic loss analysis where response history analysis is required, ground motion records need to be selected to perform the analysis. For a scenario of shaking to compare the losses of two different buildings, the two main procedures advocated are the Code/Uniform Hazard Spectra method and Hazard consistent approaches (e.g. CMS, GCIM) [32, 33]. Each has its own advantages and disadvantages which can be assessed with SQA.

The Uniform Hazard Spectra method following the code spectra is simple to conduct. However, the code is usually a simplification of a hazard found by more elaborate means and may be more severe than that from the probabilistic seismic hazard analyses (PSHA) at some periods. This can cause a large difference in the estimated demands and losses as shown by Baker [33]. Also, the dispersion in response for a particular hazard level is not obtainable.

Hazard consistent approaches (CMS, GCIM) are more consistent with PSHA demands however they have the following disadvantages: (i) There is a need for fault/recurrence data, software to conduct PSHA, and other information/ relationships, (ii) Because the hazard from PSHA is different from code spectra it may be difficult to compare the performance of the building against what it has been designed for, and (iii) They require more records and analyses than for the UHS method to consider the dispersion of response.

4.4 OTHER EXAMPLES

There are many other examples where decisions need to be made. A few research topics using this approach currently with which the author is involved at the University of Canterbury and which involve SQA include:

- Base connections (considering damageability)
- Splice types (considering construction)
- Connections to composite columns (considering cost)
- Two way connections to rectangular CFT columns (considering analysis methods)
- Flexural connections in beams away from the column (considering effectiveness and cost)

5 CONCLUSIONS

This paper treats the decision about the best type of structure for a particular situation as a constrained optimization problem considering uncertainty. The use of two methods, each with their own strengths and weaknesses, are described. It is shown that:

i) For design of individual structures, methods to develop design codes considering probabilistic techniques are not necessarily more beneficial than that of older code methods not considering probability because the calibrations of the different methods are often similar. Probabilistic methods seem to be beneficial for loss studies as they can be relatively comprehensive and include many effects and uncertainties. An example comparison between a reinforced concrete frame structure and a wall structure is shown. This provides information that may be useful to select to most appropriate building type. However, it was described that it is important for (a) all major contributions to loss to be considered, (b) sufficient reliable data to be used for input, (c) proper calibration to be done. This becomes more difficult as the complexity of the analysis increases, also resulting in increased uncertainties. Because of the uncertainties and assumptions required for such an analysis, results from probabilistic studies should be treated with healthy scepticism.

ii) Subjective Qualitative Assessment (SQA) is a simple, subjective and very approximate method. Even though it is subjective and approximate, it has some appeal to decision makers because it is simple, the user can understand it, develop it, modify it and control it. It can incorporate the outputs of probabilistic methods and it is already used in structural engineering, and in many other real life applications which require a choice between two or more different options. This method has to date not received a lot of academic attention in structural engineering.

iii) Examples of decisions relating to a number of current structural engineering topics are provided. These include choices about different building systems and analysis approaches. It can be seen that often probabilistic methods and SQA support each other. In other cases, only one method may be required.

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