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PERFORMANCE-BASED GEOTECHNICAL EARTHQUAKE ENGINEERING – HOW CAN WE USE IT IN PRACTICE?

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Abstract: *Since the publication of SEAOC Vision 2000 in 1995 the topic of performance-based earthquake engineering has been a key subject for earthquake engineers. This has principally been focussed on structural aspects of design and less so on geotechnical aspects. The framework of performance-based design was adopted some 15 years ago in the International Standard ISO 23469 for seismic design of geotechnical structures. However, very little useful and pragmatic guidance is provided for the practising engineer. The one area in which strides have been made is for the design of ports and harbours. How can we best utilise this disparate guidance for real design work? It is well understood that the seismic performance of geotechnical works is significantly affected by ground displacement. Therefore, there is a requirement to accurately model the behaviour of geotechnical structures and estimate ground displacements. This leads to concern relating to the adequacy and accuracy of computer models to estimate soil behaviour and associated settlements or displacements. This paper reviews the current state of the practice in performance-based geotechnical earthquake engineering. It looks at key questions and concerns in implementing this approach. It also provides examples of how the approach has been applied to projects including ports, bridges and offshore structures and hence attempts to provide some practical design guidance.*

Introduction

The original concept of performance-based design was discussed by Priestley and Park (1987) in relation to bridge structures. They promoted the concept of calculating the displacement capacity of substructures, based on estimates of ultimate strain capacity, plastic hinge length, and foundation conditions. Moehle (1992) later suggested a similar approach for building structures. The subsequent publication of the SEAOC Vision 2000 report (OES, 1995) brought the concept of performance-based design to the general engineering community. The key of the document is the matrix shown in Figure 1, which links a requirement to select seismic performance objectives with a level of seismic ground motion.

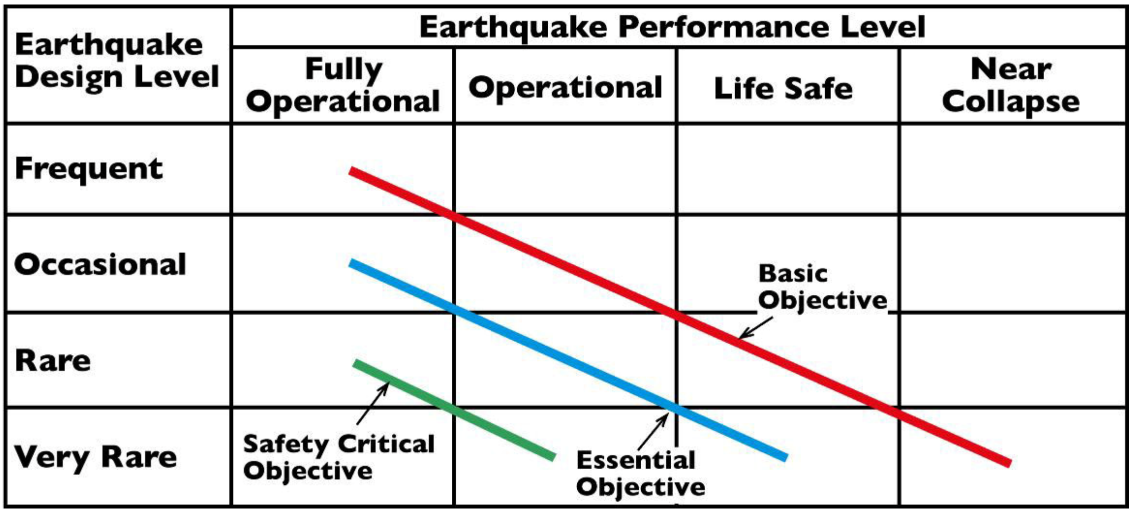

Figure 1: Performance matrix (modified from Vision 2000)

Though very elegant in concept, the qualitative nature of the different performance levels has proved difficult for practising engineers and clients to understand and use with current codes of practice (EN 1998 or ASCE/SEI 7). These performance levels mean different things to different

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people involved in the seismic design process. To a structural engineer, a measure of response such as plastic rotations in beams and columns or storey drift can be used to define building performance. To a geotechnical engineer ground displacement on a slope or rotation of a retaining wall can be a measure of performance. To a client the amount of downtime following an earthquake and the initial design and subsequent repair costs are more useful measures of performance. How these different criteria are linked together is key to the appropriate use of performance-based design.

The principle of this design methodology is excellent, but it requires a suitable framework and methodology to put it in place. A lot of work has been undertaken to assess the performance of existing structures with regard to their rehabilitation. In Europe, EN 1998-3 (2004) has provided a framework and defines acceptable strains and stresses for existing building structures, whilst in the US a more detailed framework is provided in FEMA 273 (1997) and later ASCE/SEI 41 (2017). However, it appears that much less effort has been put into the performance of foundations or geotechnical structures.

Bachman et al., (2003), as part of the ATC-58 project, provides a simple but definitive process of addressing performance-based design which is reproduced in Figure 2. It consists of selection of appropriate performance objectives, development of a design capable of achieving those objectives, verification that the design can achieve the desired objectives and, if this is not possible, iteration of the design until verification is achieved. Bachman et al., (2003) notes the importance of defining performance objectives that are both predictable, by the design professional, and meaningful and useful for the decision makers (both clients and end users) who must select or approve the performance objectives used as a basis for design.

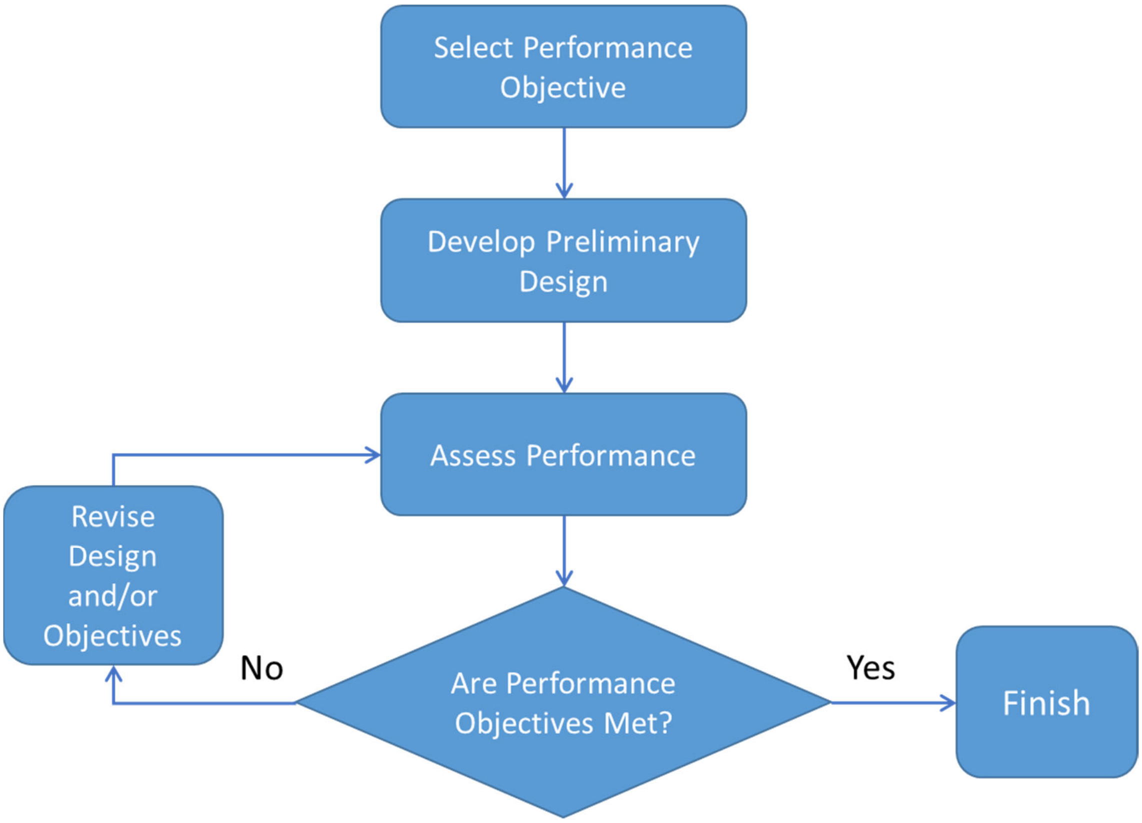

Figure 2: Performance based design flowchart (modified from Bachman et al, 2003)

This framework is simple and straightforward, but it is not enough by itself. It is essential that the performance objectives are quantifiable and easy to calculate. From a geotechnical perspective this implies defining acceptable movements or settlements for shallow foundations, slopes and retaining walls and stresses or strains for deep foundations and other structural elements. The key question for the designer and decision maker is, what is acceptable? This is the key question which, in the author's opinion, has either been ignored or sidestepped by the majority of codes and guidelines.

Geotechnical Codes of Practice

Modern seismic design codes provide a simple, economic and safe way for the design of engineering structures. The provisions are written such that they provide a minimum level of safety. Designing with modern design codes utilises the Load and Resistance Factor Design (LRFD) methodology, where the loading and resistances are factored individually with the intention of the design achieving a certain level of reliability, taking into consideration the importance of the structure or the consequences of the structure failing.

Typically, LRFD designs consider two limit states known as the serviceability and ultimate limit states, both of which are associated with a certain level of performance. Serviceability is generally associated with a level of performance that ensures, for a given level of loading, the engineering structure is not damaged and remains operable, while the ultimate limit state is associated with a less frequently occurring but more onerous loading situation, in which the structure should ensure the life safety of the occupants.

When an engineering structure is designed in this way, it is inherent through the design process that a certain level of safety is achieved, but the actual performance is not explicitly assessed. In these situations, the performance of the engineering structure in beyond design basis events is unknown. Furthermore, there are also situations where conventional LRFD approaches to design are inappropriate, such as when a geotechnical structure may be settlement sensitive (e.g. for a high-speed railway).

The following paragraphs discuss the attempts to address a performance-based design for geotechnical earthquake engineering considerations.

Dams and Reservoirs

In 1991, before performance based seismic design was being discussed, Charles et al., (1991) published guidelines related to the seismic risk of dams in the UK. This was developed from the methodology presented in ICOLD Bulletin 72 (1989). This took a first principles approach to risk by assessing the following three elements:

- The size of the primary hazard;
- The probability of occurrence; and
- The consequence of the dam breach.

Using these guidelines, the dam category (similar to the importance level in building design) is defined in terms of capacity of the reservoir, the height of the dam, the number of people at risk and the potential downstream damage, as shown in Table 1. Looking at the bracketed classification weights in the table it is clear that the biggest factors influencing the dam's category are the number of people at risk and the potential downstream damage. This contrasts with the more traditional engineering considerations of capacity of the reservoir or the height of the dam.

Table 1: Classification factors (from Charles et al., 1991)

Criterion	**Classification factors (weights)**			
Capacity (10^6 m^3)	>120 (6)	120-1 (4)	1–0.1 (2)	<0.1 (0)
Height (m)	>45 (6)	45-30 (4)	30-15 (2)	<15 (0)
Evacuation requirements (no of people)	>1000 (12)	1000-100 (8)	100-1 (4)	None (0)
Potential downstream damage	High (12)	Moderate (8)	Low (4)	None (0)

Several limiting performance criteria are provided for the safety evaluation of embankment dams. The safe shutdown earthquake (SEE) should not cause a failure of the dam which results in uncontrolled discharge of the reservoir water. The SSE should not:

- Cause liquefaction failure in the embankment or its foundations;
- Cause such a large movement on a slip surface in the slope or through the foundations that the freeboard is lost;
- Cause so much compaction of the fill or foundation that the free-board is lost;
- Cause cracks within the fill or foundation or at interfaces with structures or abutments through which uncontrolled leakage could develop;
- Damage overflow works to such an extent that dangerous conditions could develop; and
- Damage outlet works to such an extent that dangerous conditions could develop.

Though no quantitative values are presented, the designer and analyst can select appropriate values depending on the particular geometry of the embankment dam being designed or assessed.

It is somewhat disappointing that the update to the ICOLD Bulletin 72, namely ICOLD Bulletin 148 (2016), does not provide such detailed classification factors or better guidance on quantitative performance criteria.

EN 1998-5 (2004)

When initially published EN 1998-5 was lauded as one of the first codes that considered the design of geotechnical components and structures in detail (Lubkowski and Duan, 2001). In particular the consideration of kinematic interaction in pile design was literally ground breaking. However, the code is based on a force-based approach that does not allow it to be easily adapted to a performance-based approach. Furthermore, it is interesting to note there are several statements that imply the possibility of performance-based design, for example:

- For slopes clause 4.1.3.3(7) states "*The serviceability limit state condition may be checked by calculating the permanent displacement of the sliding mass by using a simplified dynamic model consisting of a rigid block sliding against a friction force on the slope*";
- For shallow foundations clause 5.4.1.1(7) states "*…a limited amount of sliding may be tolerated. The magnitude of sliding should be reasonable when the overall behaviour of the structure is considered*";
- For piles clause 5.4.2(7) states "*Piles should in principle be designed to remain elastic, but may under certain conditions be allowed to develop a plastic hinge at their heads*"; and
- For retaining walls clause 7.1(2) states "*Permanent displacements, in the form of combined sliding and tilting, the latter due to irreversible deformations of the foundation soil, may be acceptable if it is shown that they are compatible with functional and/or aesthetic requirements".*

Unfortunately, in none of these cases is there any discussion of what is acceptable or the process the designer should follow to define what is acceptable.

The ongoing re-drafting of EN1998 is attempting to take more consideration of the performance-based approach. The current draft to EN 1998-5, which is obviously subject to change, currently states "*clause 4 contains the basic performance requirements and compliance criteria applicable to geotechnical structures and geotechnical systems in seismic regions*". In principle this is very promising, however, clause 4 as it is currently written disappoints as it does not provide any guidance as to acceptable settlements, movements or strains. It discusses different consequence classes for different types of structure but fails to talk about serviceability criteria or user requirements or how a geotechnical structure should perform or be analysed in different circumstances.

For example, if one considers the design of an embankment slope the newly proposed requirements suggests a higher embankment may be more critical. At first this seems reasonable, but it is not a wholly valid argument. I would argue the purpose or risk of the embankment may be as important or more important than its height. For example, the serviceability requirements for a high-speed rail line will be far less tolerant to settlements and movement, compared to a motorway.

ISO 23469 (2005)

The framework of performance-based design was adopted some 15 years ago in the International Standard for seismic design of geotechnical structures. In the introduction it states:

- *The seismic performance of geotechnical works is significantly affected by ground displacement. In particular, soil-structure interaction and effects of liquefaction play major roles and pose difficult problems for engineers.*
- *The seismic performance criteria for geotechnical works cover a wide range. If the consequences of failure are minor and the geotechnical works are easily repairable, their failure or collapse may be acceptable and explicit seismic design may not be required. However, geotechnical works that are an essential part of a facility handling hazardous materials or a post-earthquake emergency facility shall maintain full operational capacity during and after an earthquake.*

Clause 5.1.4 discusses the philosophy of performance criteria in terms of serviceability and ultimate limit states; however, very little guidance in terms of typical criteria are provided for the practicing engineer.

ASCE/SEI 7-16 and ASCE/SEI 41-17

The US building codes for new structural design and rehabilitation of existing structures codes both provide sections on geotechnical foundation design. Though principally a LRFD code ASCE/SEI 7-16 discusses performance goals. It aims to ensure a consistent target reliability against structural collapse in an earthquake for structures of different risk categories.

From a geotechnical earthquake engineering perspective ASCE/SEI 7-16 discusses limits for foundation design within liquefied ground in Chapter 12 and permits non-linear soil-structure interaction analyses in Chapter 19, so long as sufficient time histories are employed, and uncertainty of material properties are appropriately considered.

Quantifiable performance criteria are presented, some of which are reproduced in Table 2. For example, lateral spreading limits that can be tolerated for well tied together shallow foundations and differential settlement limits to provide collapse resistance for Risk Category II and III structures. These limits are consistent with the drift limits in ASCE 41-17 to maintain collapse prevention for risk category II and III. The limits for Risk Category IV are intended to maintain differential settlements less than the distortion that will cause doors to jam in the design earthquake.

Table 2: Selected shallow foundation performance limits (from ASCE/SEI 7-16)

Criterion/Risk Category		**Upper limit for shallow foundations**		
		I or II	**III**	**IV**
Lateral spreading limit (mm)		455	305	100
Differential settlement	Single-story concrete wall system	0.0075L	0.005L	0.002L
	Other single-story systems	0.015L	0.010L	0.002L
	Multi-story concrete wall systems	0.005L	0.003L	0.002L
	Other multi-story systems	0.010L	0.006L	0.002L

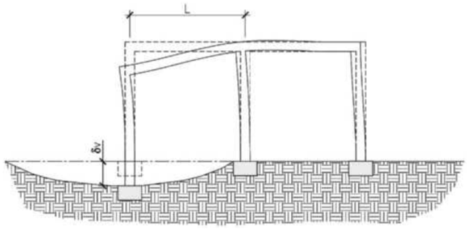

FIGURE C12.13-1 Example Showing Differential Settlement Terms δ_v and L

For piles design in liquefied ground ASCE/SEI 7-16 states "*Where nonlinear behaviour of piles occurs caused by permanent ground displacement induced by lateral spreading, the pile deformations shall not result in loss of the pile's ability to carry gravity loads, nor shall the deteriorated pile's lateral strength be less than 67% of the undamaged nominal strength*". Additional requirements are also provided, for example for concrete piles shall be detailed to meet appropriate ductility requirements "*…from the top of the pile to a depth exceeding that of the deepest layer of soil prone to lateral spreading by at least 7 times the pile diameter*".

Ports and Harbours

The maritime and ports sector is one area that has made significant strides in the development of performance-based guidelines and standards. This is principally as a response to the repair costs and associated downtime at the Port of Oakland following the 1989 Loma Prieta earthquake and the Port of Kobe following the 1995 Great Hanshin earthquake, where the poor performance of geotechnical structures was a key concern.

Apart from the direct cost of US$5.5 billion at the Port of Kobe, Werner et al., (1997) note the loss of operations at the port were estimated to be a further US$6 billion and there was a permanent 20% loss of business to the adjacent Port of Osaka. Werner et al., (2008) state earthquakes pose a significant threat to many large US seaports that serve as crucial gateways for international trade. The US Bureau of Transportation Statistics shows that 40% of the value of international trade passes through US ports and notes that maritime trade value has more than doubled since 1990 (from US$434 billion in 1990 to nearly US$900 billion in 2007) and will continue to increase. Recent history and large costs provide an incentive to improve the status quo.

The Technical Council on Lifeline Earthquake Engineering produced Monograph No 12 (ASCE, 1998) as an initial response. The committee suggested the development of recommended practices for seismic design of port structures. They concluded:

- Appropriate seismic performance requirements should be developed for critical and non-critical structures;
- Multiple levels of design for serviceability, damage control and collapse protection should be considered;
- Account for regional seismological and local geological factors in the design of ports; and
- Incorporate provisions for design against liquefaction related hazards.

The recommendation led to the development of ASCE/COPRI 61-14, which is described in more detail below.

In 2001, Working Group No 34 of the Maritime Navigation Commission developed guidelines for the seismic design of port structures (PIANC, 2001). They describe a seismic performance-based methodology which is aimed to ensure deformations of port structures fall within acceptable deformation limits depending on the nature of the structure and the importance of the facility. It requires the port owner to decide on the required performance grade, as defined in Table 3 below. The selection of the appropriate grade is dependent on the criticality of the facility considering both social and economic impacts.

Table 3: Performance grades (PIANC, 2001)

Performance Grade	**Design Earthquake**	
	Level 1	**Level 2**
Grade S	Damage Degree I: Serviceable*	Damage Degree I: Serviceable*
Grade A	Damage Degree I: Serviceable*	Damage Degree II: Repairable#
Grade B	Damage Degree I: Serviceable*	Damage Degree III: Near Collapse
Grade C	Damage Degree II: Repairable#	Damage Degree IV: Collapse
* Serviceable means little or no structural damage and little or no loss of serviceability. # Repairable means some damage and residual deformation resulting in short-term loss of serviceability.		

The guidelines suggest the Level 1 earthquake motion is typically defined as motion with a probability of exceedance of 50% during the life-span of a structure. The Level 2 earthquake motion is typically defined as motion with a probability of exceedance of 10 % during the life-span. So, if the life-span of a port structure is 50 years, the return periods for Level 1 and Level 2 are approximately 72 and 475 years, respectively.

The guidelines present acceptable damage criteria in terms of displacements, rotations or stresses for a range of port structures including pile supported wharves, sheet pile quay walls and cranes. This remains the single document that treats all the elements of a port as a single entity.

Similar criteria have been developed in Japan (OCDI, 2002), which are presented in Table 4. OCDI (2002) also provide deformation criteria for gravity and sheet pile type quay walls. It is interesting to note that OCDI (2009) appears to relax the suggested deformation criteria.

Table 4: Earthquake motion and resistance requirements (OCDI, 2002)

Ground motion level	**Ground motion return period**	**Applicable facilities**	**Earthquake resistance**
Level 1	75 years	All facilities	Do not lose their function
Level 2	Several hundred years	High seismic resistant structures (quay walls, revetments etc.)	Retain their expected function

The ASCE/COPRI 61-14 standard uses displacement-based design methods to establish guidelines for the design of pile supported piers and wharves to withstand the effects of earthquakes. Therefore, it does not consider wharves using combi-walls or large caissons. It defines suitable performance criteria as shown in Table 5, which are broadly similar to PIANC (2001) and OCDI (2002). The performance levels are defined as follows.

- For life safety, the structure shall continue to support gravity loads;
- For controlled and repairable damage, the structure should respond in a controlled and ductile manner and the required repairs should result in a loss of serviceability of no more than several months; and
- For minimal damage, the structure should exhibit near elastic structural response and there should be no loss of serviceability.

Table 5: Performance criteria (ASCE/COPRI 61-14)

<table>
<tr><th rowspan="3">Design Classification</th><th colspan="6">Seismic Hazard and Performance Level</th></tr>
<tr><th colspan="2">Operating Level Earthquake (OLE)</th><th colspan="2">Contingency Level Earthquake (CLE)</th><th colspan="2">Design Earthquake (DE)</th></tr>
<tr><th>GM#</th><th>PL</th><th>GM#</th><th>PL</th><th>GM#</th><th>PL</th></tr>
<tr><td>High</td><td>50% in 50 years</td><td>Minimal damage</td><td>10% in 50 years</td><td rowspan="2">Controlled and Repairable Damage</td><td rowspan="3">2% in 50 years</td><td rowspan="3">Life Safety</td></tr>
<tr><td>Moderate</td><td>N/A</td><td>N/A</td><td>20% in 50 years</td></tr>
<tr><td>Low</td><td>N/A</td><td>N/A</td><td>N/A</td><td>N/A</td></tr>
<tr><td colspan="7">GM – Ground Motion
PL – Performance Limit</td></tr>
</table>

Where ASCE/COPRI 61-14 takes a significant step forward is that it provides strain limits for pile supported piers and wharves. In Tables 3-1, 3-2 and 3-3 strain limits are given for different circular piles for minimal damage, repairable damage and life safety respectively, hence providing usable quantitative performance criteria. Table 6 presents a snapshot of limits from these tables for reinforced concrete piles. Apart from these strain limits there are additional stability checks that are also required by the code. The ASCE/COPRI 61-14 code also discusses the key geotechnical processes that need to be considered and the methods for undertaking displacement-based analysis and design, but these are not discussed in this paper.

Table 6: Strain limits for reinforced concrete piles (simplified from ASCE/COPRI 61-14)

Hinge location		Minimal damage	Repairable damage	Life safety
Concrete	Top of pile	$\varepsilon_c \leq 0.005$	$\varepsilon_c \leq 0.025$	No limit
	In ground	$\varepsilon_c \leq 0.005$	$\varepsilon_c \leq 0.008$	$\varepsilon_c \leq 0.012$
	Deep in ground	$\varepsilon_c \leq 0.008$	$\varepsilon_c \leq 0.012$	No limit
Reinforcing Steel		$\varepsilon_s \leq 0.015$	$\varepsilon_s \leq 0.06$	$\varepsilon_s \leq 0.08$

Linking ASCE/COPRI 61-14 with PIANC (2001) the designer can start to develop a design basis for an entire port facility from the wharf structure to the front gate, with the ability to protect the facility based on a sensible agreement with the port owner and/or operator.

Research into Geotechnical Performance-Based Design

Several leading geotechnical earthquake engineering experts have considered the issue of performance-based geotechnical design. In recent years, several Rankine lecturers have discussed the issue of performance in geotechnical engineering. The 52nd lecturer Professor Malcolm Bolton talked about performance-based design in geotechnical engineering. This was largely enabled by a huge amount of settlement and lateral movement data that is available just for a single city like London. Most recently in the 59th lecture Dr George Gazetas discussed the benefits of unconventional seismic foundation design to a very warm reception looking at real-life performance of a few structures. It is instructive to recognise that in the seismic field good quality back-analyses worldwide are extremely rare, though the situation is slowly improving.

Both lectures were highlighting the need to better understand how a geotechnical structure behaves and move away from conventional design approaches. These are valid considerations, but some questions remain largely unanswered:

- How easy are they to apply to practical design scenarios?
- Is there sufficient guidance on picking suitable performance criteria?
- Can we substantiate the analysis methods we employ?

Cubrinovski and Bradley (2009) present three different approaches for assessment of seismic performance of earth structures and soil-structure systems where deformation criteria are available. These are pseudo-static analysis, effective stress analysis and a full probabilistic framework; each are reviewed, and the benefits and limitations are discussed. There is no discussion as to what constitutes acceptable deformation criteria.

Iai and Tobita (2010) presented an overview of recent advances in earthquake geotechnical engineering with respect to the seismic design of geotechnical structures. The key conclusions drawn, in particular:

- Often problems in seismic design of soil-structure systems arise from assuming an inappropriate failure mode when using a simplified analysis;
- It is important to deal with the material, loading and analysis uncertainties when attempting to assess seismic performance and estimate associated costs; and
- Combined hazards, such as earthquake and tsunami, pose a new challenge to the geotechnical earthquake engineering profession.

Though they do not give examples of performance criteria, Iai and Tobita (2010) discuss the issue and state "*…performance criteria are specified in terms of engineering parameters that characterize the seismic response of and induced damage to geotechnical structures. The possible consequences of failure and type of analysis methods are considered in the formulation of the performance criteria.*"

Pecker et al., (2014) looked at the implementation of a design philosophy in which yielding at the foundation level is allowed for, albeit partially. They note that this requires an efficient and reliable tool to fulfil the design and propose the macro-element concept (Nova and Montrasio, 1991) to carry out non-linear soil-structure interaction analyses. Non-linear behaviour is limited to soil plasticity and foundation uplift. The results of these analyses provide a simple and reliable method of foundation performance. No discussion of defining suitable criteria is made.

Kramer (2014) highlights that assessing performance requires prediction of the response, damage and loss associated with one or more specific levels of ground shaking. To achieve this the following is required:

- A response model, which might be a detailed non-linear finite element model, is used to predict the response of a soil-structure system to earthquake shaking;
- A damage model is used to predict physical damage from response levels; and
- A loss model which in its simplest form might take the damage data to estimate repair quantities and unit costs.

Kramer (2014) reminds the reader that owners, operators, and other decision-makers are primarily interested in managing risk, so the accurate estimation of potential losses must be the ultimate goal of performance-based design procedures. Kramer (2014) shows that performance-based design offers several advantages over more traditional design approaches, and presents a simplified force based like methodology to calculate displacement demand.

Finn (2018) illustrates the key elements of performance-based design with practical examples of evaluating cost effective remedial measures for four slope stability projects in Canada, Japan and the USA. In each case three key elements are discussed, the selection of appropriate performance criteria, the selection of an appropriately validated analysis program and the calibration of the constitutive model. The Canadian example looked at developing screening criteria to assess the stability of slopes under earthquake loading. This was required because of a doubling in the seismic hazard in the British Columbia Building Code in 2005, which resulted in many slopes being considered unstable. Therefore, prime real estate was now not suitable for residential development. A study was commissioned which utilised a novel slope screening approach developed by Bray and Travasarou (2007) based on an extensive database of Newmark analyses. An acceptable slope displacement of 150mm was recommended for the screening study. The value of 150mm was selected based on experience with wood frame construction in British Columbia. This is an example when performance-based geotechnical earthquake engineering can provide an attractive alternative. Furthermore, it was shown to be robust and was simple to use for local practitioners.

Though most of these studies provide a useful and interesting insight to the application of performance-based geotechnical earthquake design, they struggle to provide clear guidance to the practising engineer. Only Finn (2018) defines criteria for slopes by consideration of the performance of the adjacent developments.

Key Analysis Principles

It is well understood that the seismic performance of geotechnical works is significantly affected by ground displacement. Therefore, there is a requirement to accurately model the behaviour of geotechnical structures and estimate ground displacements. This provides a lot of concern relating to the validity of soil models and modelling methods. The following paragraphs discuss modelling techniques, uncertainties and suitability of analysis methods.

Modelling Techniques

I often hear one of the key comments of reviewers or checkers when assessing a displacement-based analysis is "*what result does a simple pseudo-static analysis give?*". There is often a lack of confidence in the ability of a non-linear finite element model to give an appropriate result, whatever that might mean. This is somewhat frustrating as it means that engineering practice and hence our clients do not benefit from new methods and developments. This is especially seen in the nuclear industry (Lubkowski, 2015).

As a practising engineer it is important to recognise that there is no such thing as a correct analysis of a complex soil-structure system. Our role is to assess the performance of the soil-structure system using the most appropriate tools but recognise the limitations of the analysis models and calculate results (displacements, strains, bending moments) which are defensible. This does not mean we need to derive conservative results. Therefore, we need to be open about analysis limitations, and ensure we address known uncertainties, especially those which have a big impact on the result. A probabilistic framework is best suited to address this situation.

Material and Loading Uncertainties

Uncertainties in geological and geotechnical properties is caused by three key factors:

- The inherent variability of the soil or rock material;
- Errors in measurements of in-situ or laboratory tests; and
- Uncertainty in correlations or classification schemes to derive design parameters.

For example, O'Riordan et al., (2019) looked at how non-linear site response analyses are used to support performance-based design in Mexico City. This is particularly important due to the soft clays that underlie the city. The study showed that the shear wave velocities measured during the ground investigation are about 15% below the "in-situ" or "aged" condition, so low strain shear modulus is underestimated by around 30%. As an aside it is staggering how often geophysical investigation reports need to go through several iterations before you can have confidence in the results.

In addition to these, there are uncertainties related to the design process itself, including:

- Loading conditions – in-situ stress conditions and changes due to construction;
- Failure mechanisms – stress induced or dictated by the geology or combination thereof; and
- Numerical modelling methods – different constitutive models, numerical solvers, etc.

Analysis Methods

As mentioned earlier the suitability of the analysis method and associated material parameters is also important. The designer must understand the likely failure mechanism of the geotechnical structure under consideration and hence determine an appropriate analysis method. If we consider a railway embankment, would an essentially elastic analysis (e.g. Newmark sliding block) be sufficient or would an effective stress analysis be required because of potential liquefaction and associated lateral spreading? The method must be suitable for the problem being considered and the behaviour of the founding soils and any interactions with associated structures. This would tend to push you towards more advanced holistic non-linear finite element analyses, though methods such as the macro-element concept championed by Pecker et al., (2014) may also be valid.

Practical Applications

This section reviews the current state of the practice in performance-based geotechnical earthquake engineering, focussing on some special structures. It looks at key questions and concerns in implementing the approach. It also provides examples of how the approach has been applied to projects including ports, bridges and offshore structures, and hence attempt to provide some practical design guidance.

Dry Dock, Philippines

For the construction of the Malampaya concrete gravity structure (CGS) a dry dock was excavated on the shore of Subic Bay on the island of Luzon, as shown in Figure 3. The photograph is taken from the coast looking inland; the sea bund is in the foreground, with the CGS in an early stage of construction. The key stakeholders were concerned about the potential impact of an earthquake on the project following the uncontrolled flooding of the dock. It was programmed that the dry dock and CGS would be constructed during a two-year period. To design the dry dock a performance-based approach was adopted as described below.

The key issues were to derive appropriate hazard levels and then define acceptable qualitative and quantitative criteria. Once the design levels and criteria were defined appropriate analyses could be carried out to verify the suitability of the proposed design solution. Following the design, a monitoring network was also initiated to assess the performance of the internal slopes both during construction of the dry dock but also during the construction of the CGS.

Despite the relatively short 2-year construction period for the project, the extreme seismicity of the region suggested that the dock may experience earthquakes during that time. The intention of the design was to ensure the casting basin remained operational during a small event. Furthermore, the design should prevent rapid flooding of the dock, due to bund overtopping and consequent potential for loss of life in a larger rare event. The potential for movement existed due to the presence of a thick layer of loose silty sand and sandy silt marine deposits, which were believed to be susceptible to liquefaction.

It was agreed that the design of this facility should be subject to two performance levels, defined by the expected serviceability level earthquake (SLE) and ultimate design level earthquake (DLE) events described below:

- The intention of the SLE was to ensure that construction of the CGS could continue without delay or need for remedial work on the dock. A probability of exceedance of 10% was selected as this matched the value for standard design according to NSCP (1992). This led to a return period of 19 years and a surface PGA of 0.12g. Under the SLE, there was to be no liquefaction and all slopes were to have a factor of safety of greater than unity.
- The objective of the DLE was to prevent catastrophic failure of the sea bund and the subsequent loss of life due to uncontrolled flash flooding of the dock floor. Some settlement and lateral movement of the bund and slow flooding were considered acceptable, but these were to be limited such that safe evacuation of the area could be achieved. A probability of exceedance of 0.4% was selected based on comparison with shutdown criteria for a range of hazardous installations. This led to a return period of 500 years and a surface PGA of 0.41g. Under the DLE, there was to be no liquefaction and movements on the sea bund were to be limited to 200mm.

Figure 3: Aerial photograph showing dry dock and CGS construction (© Arup)

Table 7 shows the performance criteria adopted for the project. In addition, the side slopes had to achieve a factor of safety of 1.3 for static stability, following the recommendation of Johnston et al., (1999) for dams.

Table 7: Dry Dock Performance Criteria

Slope Location	**Limit State**	
	SLE	**DLE**
Sea Bund	FoS ≥1.0 when Kc ≥ 0.5 Km	Slope displacement < 200mm
Side Slopes	FoS ≥1.0 when Kc ≥ 0.5 Km	N/A

Due to the potential liquefaction, some form of mitigation was required. Considering the limited duration of the project and the need to lower the water table below the construction platform, an active dewatering system was employed to also mitigate the risk of liquefaction. Details of the dewatering design can be found in Clare et al., (2002) and will not be discussed herein. Since the risk of liquefaction was mitigated the analyses of the slopes could be relatively straightforward. They utilised standard pseudo-static methodology in Oasys SLOPE together with the Newmark sliding block approach to estimate slope movements.

Magtymguly Collector Riser (MCR-A), Caspian Sea

Gibson et al., (2012) present the performance-based seismic analyses that were carried out for a self-installing GBS in the Caspian Sea. The performance-based philosophy was adopted after it was shown that a more conventional elastic design approach predicted the platform to have inadequate capacity to withstand significant seismic events. Increasing the size of the sub-structure also increased the base shear and hence the analysis could not find an optimal solution that satisfied traditional foundation stability checks. This would have shown the GBS solution to be unacceptable.

Gibson et al., (2012) explain that a performance-based approach was adopted, in combination with advanced non-linear analyses, to demonstrate the suitability of the GBS according to appropriate project specific performance requirements.

ISO 19901-2 (2004) was adopted as the main reference for the seismic design criteria for the platform. Lubkowski et al., (2010) presents the probabilistic seismic hazard assessment used to define the seismic design criteria. The return period for the Abnormal Level Earthquake (ALE) was defined based on the exposure level of the platform and the slope of the seismic hazard curve. The return period for the Extreme Level Earthquake (ELE) was based on the ALE and a seismic reserve capacity factor (C_r) of 2.8. This was verified by a pushover analysis of the superstructure (Gibson et al., 2012).

ISO 19901-2 (2004) only provides qualitative performance criteria, so the project specific quantitative criteria shown in Table 8 were derived. These were derived based on consideration of what criterion (e.g. member forces or foundation movements) was important to achieve acceptable performance. From a geotechnical perspective sliding below the foundation level was considered an acceptable response. Therefore, the incoming pipelines and bridge piping were designed to accommodate sliding displacements to avoid rupture and environmental damage. The pipelines were configured with expansion loops to connect to the risers and allow for pipe line expansion.

Table 8: Performance levels from ISO 19901-2 (modified from Gibson et al., 2012)

Ground motion	**Performance Level**		
	ISO Description	**Structural Criteria Adopted**	**Geotechnical Criteria Adopted**
ELE (270 years)	No major damage. Limited ductile yielding acceptable. Maintain full capacity for future events	Code utilisation checks according to API RP 2A WSD (2005)	Sliding < 50mm
ALE (1,175 years)	May suffer structural damage, but integrity is maintained, and loss of life and/or major environmental damage is prevented	Plastic deformation limits to ASCE 41 (2006)	Sliding < 500mm

To accurately capture the response of the soil-structure interaction system during the ELE and ALE seismic events a finite element model of the platform and soil was built in LS-DYNA (as shown in Figure 4). Non-linear properties of the jacket, topsides structure and soils were assigned, and the adjacent well-head platform was also modelled. The uncertainty in material properties and input ground motions were also considered. This approach enabled the structural response, capacity foundation stability and relative motions between the adjacent platforms to be accurately predicted within the same analysis model.

For this study the soil was modelled using a hysteretic model LS-DYNA which followed the Masing rules (Pyke, 1979). The susceptibility of the soil to strength degradation due to pore pressure rise was considered independently and was shown not to impact the FE analysis. The low strain shear modulus (G_0) profiles and corresponding degradation curves were based on a detailed geotechnical investigation of the site, in particular cone penetration testing (CPT) data and shear wave velocity estimates from seismic cone information. The upper and lower bound estimate profiles correspond, in general terms, to 1.5 and 0.67 times the best estimate values respectively. These bounding values were taken according to the ASCE/SEI 4 (1998). As recommended in ISO 19901-2 the upper and lower bound soil properties were used for the ELE analyses and best estimate properties used for the ALE.

Gibson et al, (2012) conclude the performance-based approach adopted by the design team demonstrated the feasibility of a structure which could not have been shown to satisfy codified requirements using a conventional design approach. The alternative would have required a major redesign and a more materially intensive structure with a corresponding increase in cost. Explicitly analysing the behaviour of the platform during the ELE and ALE events improved the client's appreciation of the likely performance of their platform, enabling them to better quantify the risks involved with operating the facility.

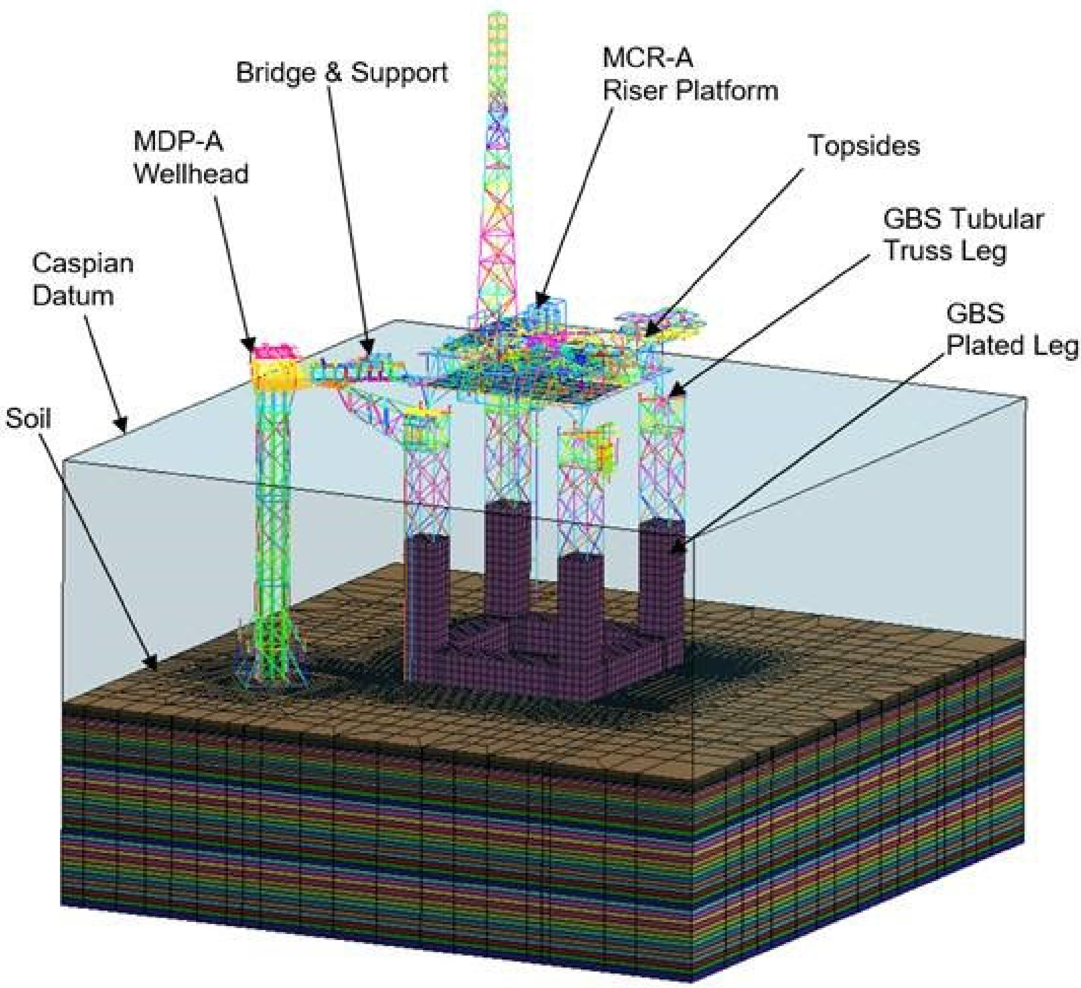

Figure 4: Overview of analysis model (© Arup)

Transbay Transit Center, San Francisco

Ellison et al., (2017) present the detailed soil-structure interaction analyses carried out for the Transbay Transit Center. This is a 425m long by 55m wide by 18m deep box excavated in a dense, urban part of San Francisco. Ellison et al., (2012) describe what they name a "rupture to rafters" model to evaluate the potential impact of two adjacent high-rise buildings on the Transbay Transit Center for the tower developers. The two buildings considered were the:

- 305m high Salesforce Tower; and
- 245m high 181 Fremont Tower.

In each case, a "rupture to rafters" analysis was utilised to provide a high level of confidence in the anticipated behaviour of the interacting structures to the seismic review panel, which allowed the design and approvals process to move forward smoothly.

Ellison et al., (2017) explain that the high-rises include basements which are founded at approximately the same depth as the Transbay Transit Center, share a shoring wall and were constructed simultaneously. Thus, there is a direct structure-to-structure load path between the adjacent buildings in the permanent condition. The resulting 3D model developed in the non-linear program LS-DYNA is shown in Figure 5. Details of the model are given in Ellison et al., (2017) and will not be repeated here.

The soil-structure interaction models for each tower were run with three bedrock ground motions selected from the larger suites of initial site response analyses. The final simulation for the 181 Fremont Tower included over 1.5 million elements and took several days of computation time to complete on twelve 2.93 GHz CPUs.

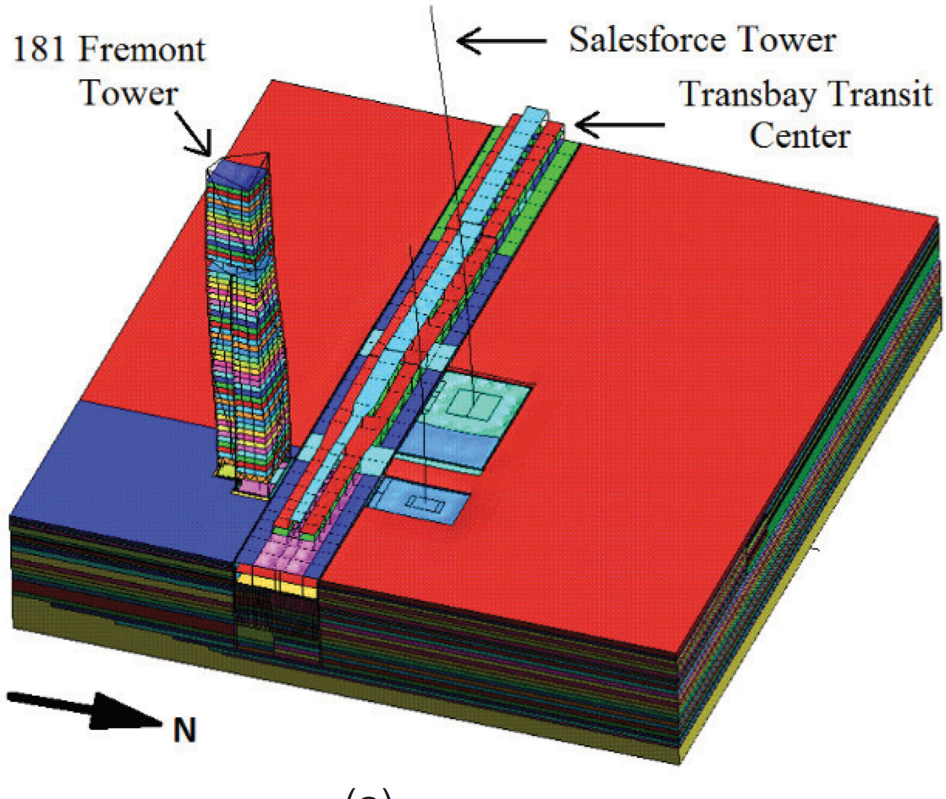

Figure 5: Model of the Transbay Transit Center and surrounding high-rises (modified from Ellison et al., 2017):

Comparisons between runs with and without the adjacent towers indicate that the seismic-induced shear forces in the Transbay Transit Center diaphragms at and below grade were elevated in the vicinity of the transverse basement walls of the adjacent towers.

In this case rather than meeting certain performance-based criteria, the analysis allowed the structural designers of the Transbay Transit Center to accommodate the additional loads from the adjacent towers. Furthermore, the analysis was able to demonstrate that the impact of the towers would not invalidate the performance objectives of the Transbay Transit Center.

Discussion and Conclusions

This paper has reviewed the state of the practice in geotechnical performance-based seismic design. It has shown the developments within codes of practice, in academic research and in practice. Several issues in implementing performance-based methods are discussed.

One of the key issues is an apparent lack of guidance in selecting appropriate performance criteria. This is not an easy issue to resolve and codify and is dependent on several factors as well as being very project and structure specific. Based on practical project experience, where a holistic approach to design has been followed, I would recommend that the following questions are considered to help define an appropriate criterion or set of criteria. These questions are:

1. Are there operational performance criteria associated with the use of the structure that may define performance limits? (e.g. rail track operational limits).
2. Are there economic considerations related to the ongoing operation of a structure? (e.g. a fixed concession period for a key infrastructure link).
3. Are there socio-economic considerations related to the structure? (e.g. the only large port within a region).
4. Are there more critical structures adjacent to the structure being assessed? (e.g. an LNG tank at the top of a slope).
5. How does the performance of the geotechnical element affect the superstructure? (e.g. the rotation of foundations for a tall bridge pier may be more important for the deck bearing than exceeding the plastic capacity of the pile).

The first four questions should be reviewed and agreed with the client, whilst the fifth should be reviewed and agreed with the designers of all other key elements. This will not only ensure the client's aims are met, but all aspects of the design work together to achieve adequate performance.

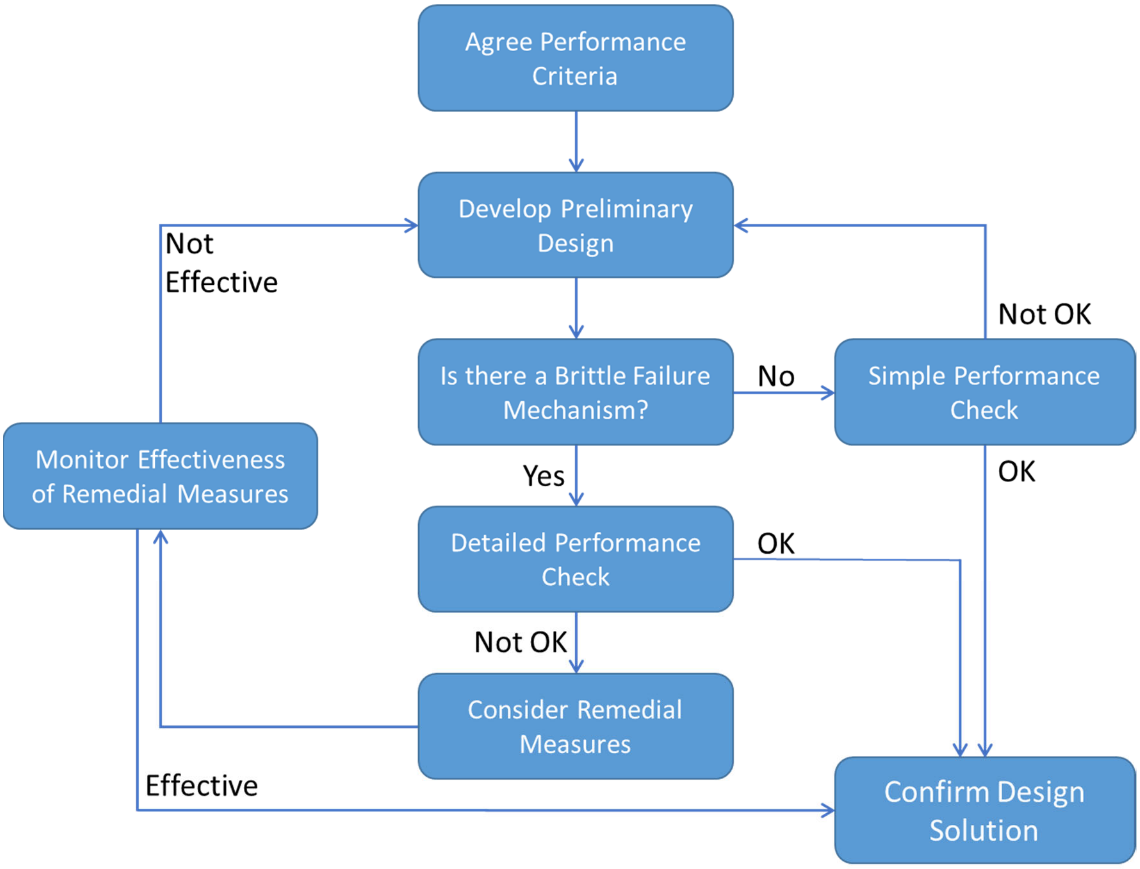

Figure 6: Simplified performance-based analysis flowchart

Once appropriate design criteria have been agreed by all parties a framework should be followed to ensure the required performance is achieved. Figure 6 is a proposed simplified flowchart that takes the intent of Figure 2 and makes it relevant to geotechnical structures. The key issue is whether the geotechnical system suffers a brittle mechanism (e.g. liquefaction, lateral spreading, slope failure etc.) and how this should be analysed and assessed. There will always be different analytical methods and preferences, the key is to select a method appropriate to the design situation.

Finally, it is essential that the engineering community gains confidence in the analysis methods we utilise and the reliability and appropriateness of dynamic soil parameters. When carrying out performance-based geotechnical calculations it is not appropriate to just use the latest "best" software, without consideration of its suitability. Validation of software and material models against real structures under significant ground motions will enable this. To this end greater deployment of vertical arrays and instrumentation of buildings in areas of high seismicity can only benefit the earthquake engineering profession and ultimately our clients.

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