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FUNKY STRUCTURE BEHAVIOR FACTORS

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ABSTRACT
Behavior (q-)factors are funky. They are fun and they are magic. Just look at how the symbol breaks the boring symmetry of a circle with a random squiggly or straight hanging tail. It perfectly embodies the spirit of the q-factor that on the surface, above the straight line of writing, seems profoundly round and deterministically predictable, while in reality it is all about the tail of unknown shape and magnitude that is hanging underneath. One may choose to ignore this, sweeping all uncertainty under a carpet of expert opinion, or attempt to directly measure it, using the best that the current state-of-art has to offer. The first option may be attractive for typical buildings, where considerable experience has been amassed, but will probably fail in misery for newer systems, interesting structures or unfamiliar situations. To capture this funky nature of the q-factor, let us try to provide a mathematically tractable definition and discuss ways of quantifying it, for a building, an ensemble of similar buildings, or a class of dissimilar buildings of the same structural system, spread over one or more sites.

Keywords: *Seismic code, behavior factor, risk, uncertainty*

1. INTRODUCTION

Let us jump straight into the fray by attempting to provide a proper definition for the optimal q-factor. To do so, we shall start by cordoning off the buildings and sites that it is meant to cover. So, let *S* be the set of all structures or structural configurations S_i to be located at site_i, $i = 1 \ldots N$, that one is interested in designing by application of a single q-factor. This could be, for example, a single structure (e.g., Prof. Adam's headquarters at Universität Innsbruck), a set of similar structures (all low-rise steel moment-resisting frames of square plan with reduced-beam-section connections), or a set of dissimilar structures of the same generic lateral load resisting system (all high-ductility class steel moment-resisting frames). Let POj, $j = 1 \ldots M$ be a number of performance objectives (POs) that each design should satisfy. Each POj is defined as a triplet of values [1]: (a) a threshold or capacity value of response, damage or loss, C, (b) a maximum allowable mean annual frequency (MAF) of exceeding this threshold, λ_O, and (c) a desired confidence level of meeting this objective vis-à-vis epistemic uncertainty, x, typically in [0.5,1). Thus, meeting an objective means that the x% percentile estimate (due to epistemic uncertainty) of the MAF of the demand, D, exceeding the capacity, C, should be lower than λ_O, or

$$\lambda_{x\%}(D > C) < \lambda_O \qquad (1)$$

Let $f(S_i, site_i, q)$ be a function that returns 1 if a code-respecting design can be found for the given structural configuration/system, site and value of q, and 0 otherwise. Then, the optimal q-factor for the set S can be defined as the largest real number $q > 1$ for which $f(S_i, site_i, q) = 1$ and all POs are satisfied for all S_i, as designed by said q. Formally:

$$\begin{aligned}&\text{Find the maximum } q \text{ to satisfy}\\&q \geq 1\\&f\left(S_i, site_i, q\right) = 1, \quad \forall i = 1,\ldots,N\\&\lambda_{xj\%}\left[D_j(S_i, site_i, q) > C_j(S_i)\right] < \lambda_{Oj}, \quad \forall i = 1,\ldots,N,\ j = 1,\ldots,M\end{aligned} \tag{2}$$

where we have defined the demand specific to each structure, site and PO_j combination in the form of $D_j(S_i, site_i, q)$ to signify that it depends (i) on the structural configuration, (ii) on the performance objective (e.g., expressed in terms of interstory drift for a damage limitation verification, versus shear force or plastic hinge rotation for a near collapse objective), (iii) on the building and the site, and (iii) on the selected value of q. Similarly, the PO capacity is expressed as $C_j(S_i,)$, as it obviously depends (i) on the structural configuration, (ii) on the PO to be checked (e.g., a drift limit versus a base shear limit). One may contend that C_j should also depend on q, since different values of capacity may apply to high versus low values of q. Think for example high versus low ductility requirements that apply to high versus low values or q in EN1998 [2]. We argue that such differentiations obviously lead to different structural configurations, consequently choosing to apply a stricter definition of S_i, where any requirements that impact the capacity limits are included directly on S_i. Thus, for example, we separate a low ductility from a high ductility version of a given structural configuration as two essentially different configurations. After all, one would probably want to employ different values of q for each.

In practice additional constraints may be placed on q. For example, rather than allowing any real positive number, q is often restricted to multiples of 0.5 (i.e., 1.0, 1.5, 2.0 and so on) for reasons of simplicity. Also, one may relax the second set of constraints, asking that the PO_j's are only fully satisfied for a certain (high) percentage of the N structural configurations that is lower than 100%, as done for example in FEMA P-695 [3]. Still, the general idea is the same: Find the largest q that produces valid designs and respects the performance objectives across all buildings and sites. Assuming we agree on this definition, let us now try to understand its implications.

First of all, we can easily argue that the fewer structural configurations S_i one needs to consider, the easier the problem becomes, and the larger the resulting q. The reason for the latter is simply that the value of q within the set of S_i is not determined by the highest performing configuration, but rather by the lowest performing one. Thus, for example, if one includes both low-rise and high-rise structures into this set, the detrimental effect of P-Delta will typically drag down the performance of taller structures and will thus set the value of q even for the shorter and better performing ones. Higher q's are generally linked to higher economy through efficient reduction of member sizes. Despite this not being necessarily true for some flexible systems, where adopting a high q may hurt economy due to drift limitations, it does lead to many well-performing systems being penalized. Conversely, this is also a problem of achieving uniform risk among different buildings, as the same buildings will achieve an even higher performance due to unneeded oversizing. Overall, the more finely one partitions the set of "all" structures into classes that receive their own q, the more uniform the seismic risk becomes between dissimilar buildings.

At the lower end, when only a single structure and site are considered, i.e., $N = 1$, the problem of Eq. (2) becomes equivalent to performance-based seismic design (PBSD). In essence, we are tasked with finding the best design that satisfies the performance objectives for a single structure, a problem that deserves a field of its own. Clearly, this will lead to the highest possible value for q for the given structure. Any other q resulting from an assessment of a superset that contains this same structure, will clearly result to an equal (at best) or lower (typically) value of q. The more similar the structures are to each other, the better our chances to get a high value. The more dissimilar they are, the more probable it becomes that we will have to lower q to make all the designs feasible.

At the opposite end of the spectrum, when N is too large, practically undeterminable or "infinite", some assumptions need to be in place to avoid having $q = 1$ become the only possible solution. This is the typical case of the design code, where all structures of a certain type to be designed in the future (e.g., all high ductility steel X-braced frames) need to share the same q. If we were to include in this set all possible irregular configurations that one may devise, a very low value of q would become inevitable. This is why, for example, EN1998 [2], specifically sets irregularity limits to the structures one may design with a given q, mandating a reduction of at least 20% if they are violated. In terms of Eq. (2), EN1998 has thus effectively partitioned the set of S_i for every structural system into at least two subsets: The regular and the irregular ones. Whether q can be determined for the latter by simply taking a 20% reduction is, I am afraid, rather debatable.

Between the two extremes of the fine-grained versus coarse-grained definition of S and associated q, one may discern a range of applicability for medium-resolution attribute-driven q-factors. Instead of splitting the buildings-to-be-designed along the boundaries of different structural systems, e.g., steel moment-resisting frames, steel eccentrical braced frames, reinforced-concrete shear walls, etc., one may employ building macro characteristics to further subdivide them into subclasses. For example, splitting any class into low/medium/high-rise categories will immediately help distinguish the detrimental effect of P-Delta on the taller buildings and allow adopting q values that progressively go down with height, rather than a single height-indifferent one. This is a similar concept to what is already done in portfolio or regional seismic loss assessment, where different fragilities are typically employed based on the number of stories.

The second major implication of Eq. (2), since it is an optimization problem that deals with nonlinear functions, such as finding the MAF of exceeding a certain PO for a given structural configuration, is that iterations are inevitable. The only possible case where this is not applicable is when enough experience exists to make a good first guess on q, and finding a perfectly optimal value is not a pre-requisite. Otherwise, an optimal q value cannot be expected to be found in a single step.

Finally, as a direct implication of the above observation, in the same way that not all assessment approaches are born equal, not all q-factors are born equal either. For the same set of S_i and $site_i$, one may estimate fairly different q-values by virtue of the approach used to assess the validity of Eq. (1). In an honest and level playing field, an assessment approach of lower fidelity would warrant a higher epistemic uncertainty, and thus be penalized with wider safety margins when determining the MAF. Consequently, it should lead to lower values of q vis-à-vis a higher fidelity approach that incorporates the state-of-art in performance assessment. Whether this is indeed done or not is a different question that is left for your consideration.

2. ASSESSMENT OPTIONS

Having a definition of q at hand, the only question left is how to assess it. There are several options to consider, all centered around five basic questions that we need to answer. Let's take a look at them, one by one.

2.1. Static or dynamic analysis?

According to a static pushover-based approach, q is usually estimated as the product of an overstrength factor and a behavior factor resulting from system ductility. The overstrength factor is $\Omega = V_{\max}/V_{\text{design}}$ according to US guidelines or a_u/a_1 as defined in EN1998-1, where $V_{\max} = a_u$ is the maximum base shear achieved on the capacity curve, V_{design} is the design base shear as estimated from the corresponding design spectrum, and a_1 is the base shear that signifies the first appearance of member plastification anywhere in the structure (e.g., the first appearance of a plastic hinge in moment-resisting frames) during the pushover analysis (Figure 2). The behavior factor due to ductility, or q_μ, is usually taken to be exactly equal to the system ductility, an assumption that is equivalent to the well-known „equal displacement rule". Thus, q_μ, is estimated as the ultimate deformation (the definition of which can be a bit problematic) divided by the nominal yield deformation, or equivalently, as the elastic design base shear divided by the maximum base shear $V_{\max}$ (Figure 1).

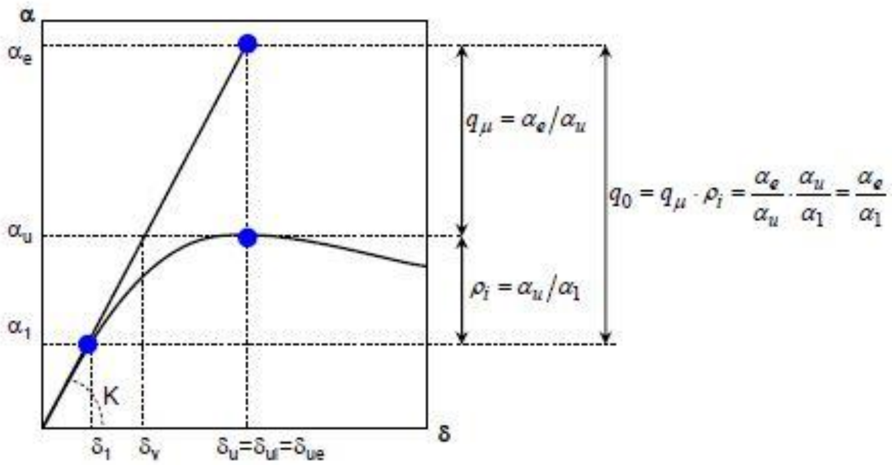

Figure 1: Definition of q in static analysis space of base shear versus roof drift (from EN1998-1 [2]).

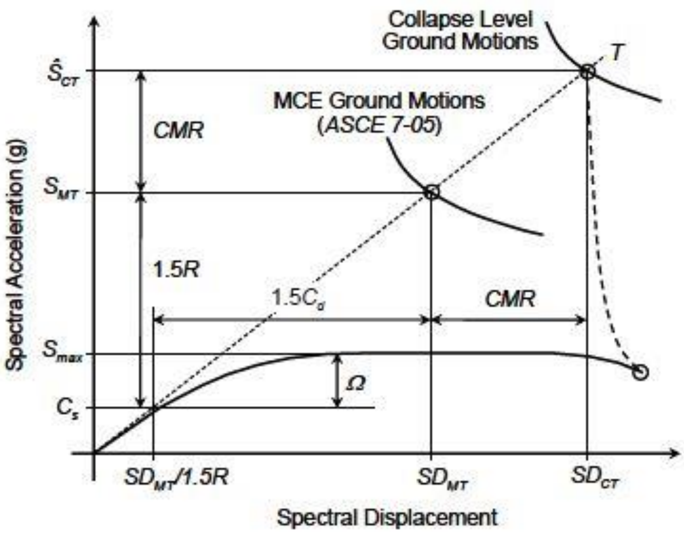

Figure 2: Definition of q based in the dynamic analysis space of seismic intensity versus building response (from FEMA P695 [3]).

The use of dynamic analysis, (e.g. FEMA P695 [3]), offers in general higher fidelity in the results, but on the same time it requires multiple accelerograms and cumbersome analysis. Furthermore, it brings along a host of new daemons related to the apparent difficulty in directly determining a value for q, whereas a pushover-based approach can seemingly do so without problems. Dynamic analyses can be employed to assess a structure (and proposed value of q) given specific performance criteria, but not for directly designing it (or determining its q). Even though some approaches for the direct assessment of q have been proposed based on dynamic analysis (e.g. Ballio et al. [5], Setti [6]) they have not been rigorously verified, and based on our earlier discussions of the nonlinear optimization nature of the problem, they seem highly unlikely to be general enough. This leads us to our next question to investigate.

2.2. Direct or indirect assessment?

Either done statically, or dynamically, the assessment of q can take two different paths: (i) a direct approach, by defining a value of q based on the results of the analysis of a set of archetype structures, or (ii) and indirect approach whereby assessment for specific performance criteria is employed to verify the suitability of a proposed q-factor. The two methods differ fundamentally, both in their practical application and in their philosophy.

From a practical point of view, in the case of direct assessment, the usual practice is not to go into iterations of redesign and reanalysis, especially if the q value initially used for design does not differ “considerably” from the one estimated in the end. In contrast, using a verification approach invariably leads to iterations of design and assessment, as we cannot estimate a q-factor, but only receive a yes or no on whether a proposed value is adequate. If the answer is yes, then the q-factor can be increased if the answer is no, it surely needs to be decreased. After any change of q, we need to repeat the process and reassess the resulting performance for each archetype structure, until the value of q employed in the current design step does not need to change in any direction, typically by borderline satisfying the performance criteria. Undoubtedly, this is a much more costly approach compared to the tradition of direct assessment.

From the view of application philosophy, the verification approach is a clear winner, as it transparently links the estimated value of q with the satisfaction of specific performance objectives. This is the main reason for using in in the FEMA P695 [3] guidelines. The direct assessment approach, may be simpler in its application, but at the same time it is based on gross assumptions of dubious reliability. Typically, all methods allowing the direct assessment of q employ rough statistical observations (e.g., the rule of equal displacements) or empirical observations that are not widely applicable to all types of structural systems regardless of their ductility capacity. In other words, the illusion of “direct assessment” needs us to take a leap of faith, disengaging from a strict scientific basis of quantitative assessment of q. This cannot easily result to a uniform level of safety for any structural system, be it brittle or ductile that may be susceptible to any different kind of failure mechanisms.

2.3. Which intensity measure?

A less obvious question concerns the selection of an appropriate intensity measure to be employed for the assessment of q. Most methodologies tend to make the same classical choice of the 5% damped first-mode spectral acceleration, $Sa(T_1)$. This is practically the only available choice for pushover-based approaches, and it brings with it all the problems of the nonlinear static approximation. Specifically, it ignores higher and elongated modes (due to plastification). Furthermore, when dynamic analysis is employed it requires relatively large scaling factors to achieve global collapse and, unless proper record selection is employed, it needs a correction for their spectral shape to remove any bias in the estimation of q [3].

A recently proposed intensity measure that has been in the works for nearly two decades is *AvgSa* [7–13]. This is the geometric mean of multiple spectral ordinates S_a (all 5% damped) estimated at periods T_{Ri} (i = 1…n) that can be chosen to characterize the entire class of archetype structures under investigation:

$$AvgSa\left(T_{Ri}\right)=\left(\prod_{i=1}^{n} S_a\left(T_{Ri}\right)\right)^{1/n} \quad (3)$$

Every value of S_a in Eq. [3] is actually the geometric mean of the corresponding spectral acceleration values of the two horizontal components of the ground motion rather than an arbitrary selection of one of them. The T_{Ri} periods may be selected at equal spacing within the range defined by a low period T_L, which represents the second or third eigenmode, and a higher period T_H, which corresponds to an elongated first mode. *AvgSa* brings multiple benefits, offering low bias [8,10, 12], low dispersion in the response [7–13], as well as low scaling factors to achieve collapse. When combined with hazard-consistent record selection, it offers a superior approach to determining performance, as well as a concrete basis for comparing different reincarnations of a given archetype.

2.4. How many and which limit-states?

The use of a verification approach offers the capability of employing any number of limit-states, and associated performance objectives that we wish our structures to satisfy (e.g., see Figure 3). In general, FEMA P695 [3] only employs the global collapse (or collapse prevention) of the structure, while EN1998-1 mainly addresses life safety, claiming to offer assurances (but without any quantitative evidence) on satisfying collapse prevention as well. A q-verification methodology that is compatible with EN1998-1 offers the option to incorporate both of these limit-states into the assessment, so that q-factors that provide both collapse prevention and life safety can be determined, offering the required level of safety with quantitative evidence, rather by implication. Obviously, any number of additional performance objectives may be added at will to meet the needs of the code, or the client.

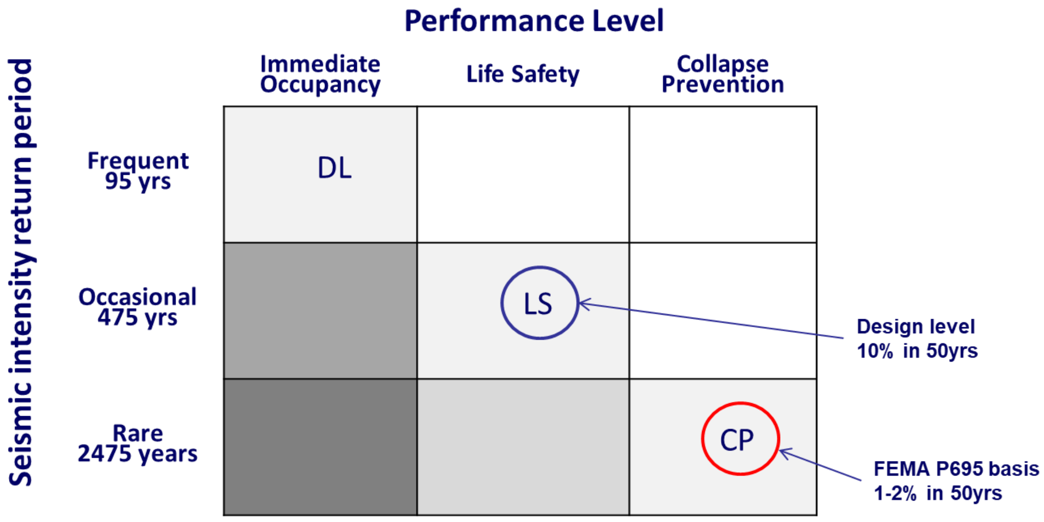

Figure 3: Potential limit states for the determination of q factors.

2.5. Intensity or risk based?

The indirect approach of verification essentially forces us to define the basis on which the said verification approach will be effected. Specifically, the FEMA P695 method verifies q via an intensity-based approach. Even though there is probability instilled in this method, in reality its use hinges on having available risk-targeted spectra, which at the moment are only employed in the US. Even having them available, though, still does not make this approach a full probabilistic solution, as it contains non-negligible errors. On the contrary, a verification method based on risk assessment has no such issues and offers unparalleled accuracy at small computational overhead. Therefore, it is considered as advantageous, and actually it is the only feasible approach for Europe until risk-targeted spectra become a part of EN1998-1.

3. CONCLUSIONS

The one schlussfolgerung that you need to take home is that q-factors may be funky, fun and magic little numbers, but they do not have to be hidden in the shadows of empiricism. Just like most other engineering problems, once a consensus is reached on their definition, they can be treated quantitatively, and in this case by using already available tools [13]. Then, that accompanying pesky tail of uncertainty that seems to create so much trouble, can be tamed to serve our noble purposes of achieving safety and economy for future structures.

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