NZ LOADINGS STANDARD (NZS1170.5) 2016 AND 2018 MODIFICATIONS TO STRUCTURAL CLAUSES FOR INCREASED SEISMIC RESILIENCE

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Abstract

The New Zealand Standard for Structural Design Actions, NZS1170.5 has required amendment after recent earthquakes in New Zealand.

The September 2016 amendment (Amendment 1) involved making modifications for a number of issues including the spectral shape factor, allowances for subsoil amplification, spectra for vertical loading, overlap distances for ramps/stairs between building floors or adjacent buildings, floor diaphragm design, non-structural elements, and ratchetting of structures. The definition of Ultimate Limit State was also amended. Many of the revisions were in response to the recommendations of the Royal Commission on the Canterbury Earthquakes.

After the Kaikoura earthquake in November 2016, the New Zealand Government requested that any issues with the existing standard be considered and addressed in a further amendment likely to be available in 2019. This addresses ratchetting provisions, which have been both simplified and generalised, as well as building inelastic torsion effects. The amendment committee was also requested to consider changes as a result of the partial collapse that occurred to the Statistics Building and in particular changes in relation to the effects of the Wellington Basin.

Also, it may be of interest to overseas engineers that the current NZ standard does not yet explicitly address the MCE level of earthquake shaking.

This paper describes the provisions incorporated, or likely to be incorporated into the 2016 and 2018 amendments, the reasons for them, and some of the latest thinking on these topics. In addition, the method by which the standard considers levels of shaking greater than the design level, and reasons both for and against this approach, are described.

Introduction

Sizable earthquakes in New Zealand over the last eight years have provided the impetus for inclusion of several new, and the review of a number of existing, requirements for seismic design of new buildings in New Zealand.

The Canterbury Earthquakes Royal Commission deliberated following the Canterbury Earthquake Sequence (which included the damaging 22 February 2011 Christchurch Earthquake) and made over thirty recommendations for action that could be considered relevant to definition of design earthquake loadings in New Zealand. These covered issues such as; specification of vertical earthquake effects, allowances for drift, definition of ultimate limit state for earthquake, allowances for regularity (torsional and unbalanced strength), diaphragms and provisions for support of stairs and ramps.

The damage that occurred in Wellington City buildings during the 14 October 2016 Kaikoura Earthquake has led to a further review of design requirements for new buildings including allowances for amplification of loadings due to basin effects within Wellington Harbour.

These reviews have culminated in two amendments of the New Zealand loadings standard, Amendment 1, issued in 2016, and Amendment 2, which at the time of writing is in the process of finalization and expected to be available in May 2019. The intention has been to undertake a full review of the loadings standard in the near future and therefore the scope of these amendments has been set with this in mind and are interim in nature. A wholesale review that could well result in a complete change to the way design earthquake loads are derived and the limit states that need to be considered was considered out of scope for these particular amendments.

Although the scope of the review of the Standard has been limited, the opportunity has been taken in Amendment 2 to also improve provisions relating to ratchetting, parts and components and torsional stability of ductile buildings.

Design Loadings

A review of design loadings following the Canterbury earthquake sequence has led to the following changes in design actions/demands:

- An increase in the Hazard Factor, Z, for the Canterbury Earthquake Region from 0.22 to 0.3 to reflect an expected heightened level of seismic activity over the next few decades after which it is expected the activity will return to pre-2010 levels.
- Consolidation into the Standard of the minimum values for Northland that were previously cited in the Building Code.
- Ability to interpolate between site subsoil classes C and D, thereby removing the potential large increase in actions resulting from moving between these classifications.
- Introduction of a hierarchy of methods for site classification.
- Changes to the vertical hazard spectra. These are discussed further below.

Following the Kaikoura Earthquake and significant damage to relatively new multi-storey buildings in central Wellington, consideration is being given to incorporating provisions to account for amplification effects for sites within the Wellington basin. It is of interest to note that the potential area affected includes the area that drove the request to allow interpolation between site subsoils C and D. For that area any basin effect is likely to be significantly greater than any relief that interpolation might have brought.

At the time of writing possible options for incorporating a Wellington basin effect and the potential impacts are still under consideration.

Torsional Stability of Ductile Buildings

It had been recognized for some time that the code provisions for allowing for building eccentricity, which have been predominantly elastically based, may not be sufficient for some buildings once inelastic behaviour occurs.

In the Canterbury earthquake sequence the damage to a number of buildings, which would typically be described as regular, could be explained by torsional instability resulting from the non-uniform development of inelasticity in parallel, otherwise identical, lateral load resisting elements. With increasing earthquake shaking, damage resulted in an ever increasing level of plan irregularity and greater levels of plan rotation than would have previously been expected or allowed for in the design for buildings of this type.

The effect was very apparent in a rectangular multi-storey building with near identical ductile frames on the perimeter of opposing sides of the building. On the basis of an elastic code based analysis the torsional resistance was predominantly provided by the frames which were spaced the greatest distance apart. When the earthquake loading was in the direction of these frames a small irregularity was surmised to cause one of the frames to enter the inelastic range before the other. The loss in stiffness in the inelastic frame caused the centre of rotation to move towards the undamaged frame increasing the torsion on the building which could not be resisted by the couples involving the yielding frame and the perpendicular frames. As a result the building had become torsionally irregular and the ductility demands on the yielding frame and the demands on the perpendicular frames would have been much greater than would have been calculated using the then code provisions.

Amendment 1 of NZS 1170.5 requires increased demands due to inelastic torsion to be considered in the design of buildings where the design ductility is greater than 1.25 and there are fewer than three major lines of resistance in the direction being considered. It is felt that three lines as a minimum provides some assurance that torsional resistance can be maintained when inelastic behavior is experienced. Structures not meeting these requirements need to be subjected to further consideration of inelastic torsional effects in the design although it is recognized a full appreciation of these effects needs further research.

Further provisions are under discussion at the time of writing.

Ratcheting

Structures that are required to exhibit ductile behavior in an earthquake and with a significantly greater strength in one direction from the other have the potential to be subjected to higher lateral inelastic displacements in the weaker direction. These inelastic deformations can increase progressively over the duration of strong earthquake shaking and can lead to accumulated deformations in the weaker direction that are significantly greater than might be predicted by elastic based methods, typically employed in design. These additional deformations can have ramifications for seating of secondary structural and non-structural elements and for the stability of affected structures.

The phenomenon is referred to as ratcheting and made its first appearance in NZS 1170.5 in Amendment 1 following evidence that the behaviour had affected several buildings during the 2011 Christchurch Earthquake.

The potential influence of ratcheting is assessed by calculating a ratcheting index which is basically the ratio of the provided lateral strength in the strong direction to the strength in the weak (reverse) direction. If the ratcheting index is above a certain level, dependent on the expected hysteresis shape for the structure, then the deflection profile for the structure in the weak direction is determined by multiplying the deflection profile in the strong direction by a ratcheting magnification factor.

The effects of ratcheting can be mitigated by a designer by taking steps to ensure the strength provided is similar in each (forward and reverse) direction of loading irrespective of the determined demand.

The degree of out of balance that requires action and the extent of the amplification required is currently under review for the latest amendment of NZS1170.5.

Diaphragms

The determination of actions in diaphragms and their design has only relatively recently received the attention it deserves in New Zealand notwithstanding that the issues have been recognized for some time. The widespread use of precast concrete systems in New Zealand and the observed behaviour of these in the recent earthquakes and testing has confirmed the importance of diaphragms in achieving earthquake resilience in buildings.

Recent additions to the Standard in relation to resilience of diaphragms include:

- Clarification that diaphragms are to be considered as part of the primary structure requiring specific consideration of continuity of load paths to primary lateral structure. In the past there has been confusion regarding whether or not diaphragms are secondary structure and therefore whether or not the actions specified in the Parts and Components section (refer below) of the standard applied.
- Consideration of effects from local deformation of primary lateral structure, including at active links of EBFs and extremities of shear walls.
- Specific consideration of deformations resulting from inelastic behaviour of primary lateral structure e.g plastic hinge elongation effects in ductile moment resisting frames.
- Consideration of actions induced by deformations of supporting structure e.g. hogging and positive induced moments at the supports of hollowcore floors.
- Importance of diaphragm collector elements and discontinuities in the diaphragm configuration.
- Method for evaluating diaphragm actions, i.e. using the pESA distribution of diaphragm forces and overstrength of the primary lateral structure.

Although it can be argued that many of these aspects are not new, specific coverage in the Standard should lead to better treatment of these elements and as a result more resilient structures.

Parts and Components

The recent earthquakes in New Zealand have supported observations from elsewhere that damage to secondary structural and non-structural elements (referred to as parts and components in NZS 1170.5) can have a significant impact on the ability to occupy buildings following even a moderate earthquake event. Historically in New Zealand these elements have not typically been well designed for earthquake effects, notwithstanding that they have been included in the earthquake loadings standards for some time.

In NZS 1170.5 parts of buildings and components are categorized in terms of the risk they pose and the degree to which they are required to remain functional or operational following an earthquake event.

Amendments have been made to the Standard to:

- Clarify that any part or component can fall within multiple categories and should be designed to meet the most restrictive of the requirements for any applying category. This was an important clarification as it had been observed that prior to this elements such as suspended ceilings had been incorrectly classified as not requiring ULS design with the result that many are understrength and suffered damage in the recent earthquakes.
- Clarify what parts or components require positive lateral support. Minor elements of low mass that can fall limited distances are not required to be restrained for earthquake forces.

- Introduction of category for parts required to maintain operational/functional continuity in all buildings, not just those classified as post disaster structures. This is in recognition that there are some elements that are necessary for commercial and residential buildings to remain occupied and that the occupiable state is necessary at levels of shaking greater than that currently defined for the onset of damage. Currently this requirement only applies for parts and components but it is likely that it will constitute a new limit state for the structure of these buildings (between current serviceability limit state, SLS1 and the ultimate limit state, ULS) once a more detailed review of the Standard has been completed.

Parts Supported on Ledges

Complete failure of a precast concrete scissor stair in an 18 storey building during the 22 February 2011 Christchurch Earthquake (CERC Volume 2) indicated both the importance of maintaining stair access in high rise buildings to enable egress after an earthquake and the particular vulnerability of stair cases reliant on ledges for support. In this particular case the stair support ledge was arguably sufficient to sustain the building lateral deformations predicted during the earthquake but it has been surmised that permanent deformation in the flight when forced into compression as clearance gaps were taken up, shortened the flight so that on the reverse cycle the provided ledge length was insufficient and the flight(s) fell taking out the complete stair.

This failure indicated the need for sizing ledges conservatively for parts of buildings where these are the sole means of vertical support and where exceeding the ledge length results in collapse. It was also recognized that in order to meet safety objectives vertical support provided by such ledges must be capable of sustaining typical design deformations by a considerable margin and that these should not be reduced for ductility in the structure via the structural performance factor, S_p.

The provisions now require ledges to be sized to cope with ULS drifts multiplied by $2/S_p$ after allowing for all other factors that could lead to a reduction in support length such as construction tolerances, creep and shrinkage, foundation deformations, spalling and permanent inelastic deformations in the part between the points of support.

Ultimate Limit State Definition

In the 1992 Earthquake Loadings Standard (NZS 4203) it was recognized that Ultimate Limit State (ULS) for earthquake represented a different limit state (greater levels of strain) than applied for other load cases, e.g. gravity and wind.

This distinction was lost in NZS 1170.5 in the attempt to reach consensus with the Australians regarding a joint standard for general loadings, including earthquake. A definition referring to a state of instability, losing equilibrium and a small residual capacity to prevent collapse resulted. This caused confusion over what the ULS for earthquake represented and how much margin was expected against collapse once the ULS had been reached.

The definition has now been amended to relate to strength, strain, ductility and deformation limits specified for the ULS in the Standard and a for reserve capacity (deliberately undefined) to avoid structural collapse, even though the structure may have sustained significant structural damage.

Definition of Requirements for Shaking Beyond the ULS

Although it is generally accepted that to meet the seismic objectives set out in clause B1 of the NZ Building Regulations (1992) there is a need for structures to perform satisfactorily in earthquake shaking beyond the levels defined for the ULS, the means necessary to achieve this have led to some considerable debate.

In the main the debate appears to revolve around whether or not an additional limit state using an MCE level of shaking (Maximum Considered Earthquake) should be defined and designed for. Typically what has been suggested is that a level of shaking approximately 1.5 times that defined for the ULS should be used. What is less clear are the criteria that would be used to check compliance for such a limit state and how these would fit within the risk based framework of the New Zealand Building Code.

Those proposing an MCE limit state cite the need to design for greater deformations than have been typically designed for at the ULS and raise the need to consider peak deformations rather than what they believe are average deformations calculated using the current methods specified in the Standard.

The counter view does not appear to dispute that buildings need to be able to sustain deformations higher than those calculated from ULS levels of shaking, but disagree on how to achieve this and the explanations made for it. The following arguments are advanced in support of this position:

- MCE is an artificial level of shaking, perhaps no different from ULS, although obviously larger.
- If an MCE limit state is to be defined, what level of shaking should it be at and what level design performance is expected for it?
- Defining any level of shaking as a maximum sends a poor message to those who have traditionally assumed that structures need not survive greater than ULS defined shaking with obvious deficiencies in resilience the result. These deficiencies will still potentially be present, albeit at a greater level of shaking. Will this be sufficient?
- To achieve the performance objectives of the Building Code requires structures to have a realistic chance of surviving earthquake shaking much greater than 1.5 times ULS shaking. Defining and understanding what constitutes “realistic” in these terms is a crucial part in understanding what level of reliability is required at these very high levels of shaking.
- Another limit state introduces additional complexity into the design process which may not achieve its objective, unless it is well considered.
- The displacements currently calculated in NZS 1170.5 for the ULS are determined using the code defined demand but are design displacements. They have neither the connotation of peak or average values but are values that when used together with all other code requirements and the ULS levels of demand are intended to provide confidence that a structure will be able to perform to the required levels of reliability across all levels of earthquake shaking to deliver the requirement level of performance overall.
- If structures need to be designed for higher levels of deformation to provide this confidence then this can be introduced at the ULS. For example the additional requirements for ledge lengths introduced to address a particular issue. These are deliberately set conservatively at ULS rather than the suggestion that they are at calculable maximum level.

There is no right or wrong way to address these issues but it is important that the objectives are clearly understood by designers and any additional design effort can be justified in the context of providing resilience within the risk-based code framework. Consideration of these aspects will be a necessary part of any future general review of earthquake design provisions and of the New Zealand Earthquake Loadings Standard in particular.

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