STRAIN HARDENING OF REINFORCEMENT IN CONCRETE BUILDINGS DURING EARTHQUAKES

A P RUSSELL
Cement and Concrete Association of New Zealand (CCANZ)

SUMMARY

Reinforced concrete moment resisting frames performed in a generally acceptable manner in the Christchurch earthquakes in terms of life safety, although many experienced damage. A phenomenon has been identified where the extent of damage and viability of repairing that damage has come under scrutiny as a result of strain hardening of the steel reinforcement due to the imposed deformations.

Where any strain hardening has occurred in the reinforcement of a concrete building, the view could be taken that there is insufficient strain capacity remaining for the building to be considered as in the same state as it was before the earthquake. This disguises the fact that different reinforcement types and grades have different strain characteristics and the onset of strain hardening does not necessarily imply that insufficient strain capacity remains. Insurers have recently become aware of this issue and are monitoring concrete buildings affected in Christchurch. One possible outcome is that insurance premiums for all concrete buildings could increase as a result of this issue.

It is important that designers and insurers are aware of the details of reinforcement used in New Zealand both currently and historically. This paper outlines the characteristics of New Zealand reinforcement, and what strain hardening implies for building performance both during an earthquake and post-event.

BACKGROUND

Reinforced concrete buildings inherently rely on the ductile characteristics of steel reinforcing to ensure a predictable and non-brittle response when subjected to loads. This is especially important for earthquake loading and where capacity design is utilised. Steel reinforcement generally is designed to behave elastically during service loading, and to yield at ultimate loads. When the yield strain (corresponding to the yield stress) is exceeded, irrecoverable plastic deformation occurs, whereas the elastic portion of strain is recoverable. As deformation increases following yielding, the strain in the bar will increase but the stress remains constant, and this is known as the yield plateau.

As the strain in steel reinforcing increases beyond the yield plateau, strain hardening occurs and the remaining strain capacity becomes less. Following deformation of a steel reinforcing bar, such as during an earthquake, testing can be performed to determine the amount of strain hardening which has occurred in the bar, and in Christchurch this is being correlated to determine the accumulated plastic strain, and thus the percentage of peak strain capacity remaining. When peak strain is reached, the bar will rupture.
Insurers are becoming concerned about a trend to cite strain hardening as a reason for demolition of damaged reinforced concrete buildings in Christchurch. Similarly, the Engineering Advisory Group (EAG, 2013) of the New Zealand Structural Engineering Society (SESOC) has noted that an owner’s insurance policy may entitle the owner to a level of reinstatement that is not practically achievable without full or partial replacement of the building structure. Whilst there is continuing debate as to whether it is the interpretation of the insurance policy document or the interpretation of the technical assessment and structural analysis that is being used as cause for demolition of some buildings, the issue of strain hardening and potentially diminished plastic deformation capacity nevertheless requires consideration.

The EAG further notes that:

“The impact of damage on future performance is dependent on the nature of the damage, and the extent to which it may be repaired or the structure modified. One of the difficulties of this assessment is that while the capacity of the building may be essentially unchanged by the damage, there is a possibility that the future performance may be reduced as a result of the number of cycles of loading sustained during the earthquakes. This applies mainly to potential strain hardening of the reinforcement.”

The correlation between the surface hardness of a steel reinforcing bar and its capacity to further deform in a ductile manner is highly complex. Moreover, the correlation between the characteristics of steel reinforcement in a concrete moment resisting frame (CMRF) and the overall seismic performance of the building similarly involves considerable complexity.

NEW ZEALAND REINFORCING

The types of reinforcing steel that can be used in New Zealand are governed by a joint Australian/New Zealand Standard, AS/NZS 4671:2001. This Standard outlines a number of types of reinforcing steel, of which two are most commonly used in New Zealand. These are Grade 300E and Grade 500E and the primary difference between the two types is the yield strength of the steel. Grade 300E reinforcement has a nominal yield strength of 300 MPa and Grade 500E reinforcement has a nominal yield strength of 500 MPa. The different types of reinforcing steel can be identified by the different bar marks rolled into them. Bar marks used by Pacific Steel are shown in Figure 1 for deformed bars and Figure 2 for plain round Grade 500E bars. Plain round Grade 300E bars are identified by the absence of any bar mark.

Figure 1: Bar marks identifying Grade 300E and Grade 500E reinforcement

Figure 2: Bar mark used to identify plain (non-deformed) Grade 500E bars
Like other materials the actual strength of reinforcing steel is not likely to be the same as the
nominal or specified strength. As with concrete, the nominal strength of 300 MPa or 500 MPa
is a lower characteristic value, which indicates that theoretically 19 out of 20 samples are
stronger than the nominal strength. Although it is difficult to put a precise value on the
average strength, for Grade 300E reinforcement the average yield strength is around
320 MPa, and for Grade 500E reinforcement it is around 550 MPa.

The ‘E’ suffix indicates that Grade 300E and Grade 500E reinforcement are both high
ductility steels suitable for use in earthquake resistant structures. Despite the similarity of the
designations there are quite significant differences (other than strength) between the two
reinforcement types. These are summarised in Table 1. A third type of steel, Grade 500N, is
also listed in the table. This is available in New Zealand and may be used in limited
circumstances. It is shown mainly for comparative purposes to illustrate indicative properties
of reinforcement unsuitable for use in earthquake resistant structures.

<table>
<thead>
<tr>
<th>Grade</th>
<th>Yield strength</th>
<th>Ratio $f_u/f_y$</th>
<th>Strain at $f_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5%ile</td>
<td>95%ile</td>
<td>minimum</td>
</tr>
<tr>
<td>300E</td>
<td>≥300</td>
<td>≤380</td>
<td>1.15</td>
</tr>
<tr>
<td>500E</td>
<td>≥500</td>
<td>≤600</td>
<td>1.15</td>
</tr>
<tr>
<td>500N</td>
<td>≥500</td>
<td>≤650</td>
<td>1.08</td>
</tr>
</tbody>
</table>

Table 1: Requirements for reinforcing steel according to AS/NZS 4671:2001

Figure 3 shows the stress-strain curves for reinforcing steel used in New Zealand.

![Stress-Strain Characteristics of New Zealand Reinforcing Steel](image)
It has generally not been important for engineers to understand the manufacturing process for reinforcing steel. However, in recent years one aspect of the process has become important since it affects the properties of Grade 500E reinforcement relevant to design. This is because the strength of mild steel can be increased in two different ways.

In New Zealand, high strength reinforcing steel has traditionally been produced by adding alloying agents (especially vanadium) to the steel melt. These alloying agents alter the structure of the steel and provide the additional strength. Importantly, the metallurgic properties of bars produced from micro alloy steel are uniform across the cross section.

Internationally high strength reinforcing steel is almost always manufactured by quenching and tempering the bar. The process used is to rapidly cool hot reinforcing bar (quenching), which causes the outer part of the bar to harden. The residual heat in the core of the bar tempers the outer case, resulting in a ductile bar with a hardened outer case and a softer pearlite core. Figure 4 shows an obviously non-homogeneous cross section of a Q+T reinforcing bar. Reinforcing steel imported into New Zealand is normally manufactured by the Q+T process, and for a short period of time before March of 2013 Pacific Steel manufactured Q+T reinforcement as well.

![Figure 4: Etched cross section of a Q+T reinforcing bar](image)

For reasons discussed in the following section it is vital that engineers are aware of the type of reinforcement being used in their projects. Pacific Steel clearly mark whether Grade 500E reinforcement is manufactured by the Q+T or micro alloy process as shown in Figure 5. Reinforcement from other suppliers may not be so clearly labelled.

![Figure 5: Printed labelling used to differentiate MA and Q+T reinforcing bars](image)

**Limitations of Q+T reinforcement**

While Q+T reinforcement can be manufactured so that it meets New Zealand Standards and is cheaper than high strength micro alloy bars, there are significant reasons why it should be considered differently.
The non-homogenous nature of Q+T reinforcement means certain restrictions are placed on its use:

- Q+T reinforcing bar must never be heated above ~450°C. If the temperature of the bar does exceed this level, the properties of the hardened outer layer will revert to be the same as the core. This will cause local weak points to form in the bar, potentially leading to failure during earthquake loading. This requirement means Q+T reinforcement can never be welded, and nor can it be rebent.

- Cutting a thread in a Q+T rebar will yield different results than occur with homogenous reinforcement. A Q+T bar gains more of its strength from the hard quenched casing. Therefore cutting a thread into this outer casing will mean that the strength reduction is not proportional to the amount of steel which is removed.

Q+T reinforcement is a fundamentally less robust material than either micro alloy Grade 500E reinforcement or Grade 300E reinforcement. Great care must be taken if using Q+T reinforcement, particularly if the reinforcement has been imported from an overseas mill that may not have the same level of quality control as New Zealand manufacturers.

**Historic reinforcing steel types**

Although only two grades of reinforcement are used in New Zealand today, other types of reinforcement have been used in earlier times. While knowledge of these reinforcement types is irrelevant to the designer of new structures, it is vital that engineers understand older reinforcement types when assessing the performance of existing buildings. Some of the more prevalent previous reinforcement types are described below.

**Grade 275** Before 1989 New Zealand reinforcement was graded according to the “minimum” yield stress, rather than the lower characteristic yield stress as is used now. The steel used in Grade 300E reinforcement has a minimum yield strength of approximately 275 MPa, so before 1989 this reinforcement was referred to as Grade 275.

**Grade 430** Grade 430 reinforcement was the predecessor to Grade 500E. It was a high ductility micro alloy steel with a lower characteristic yield strength of 430 MPa. Grade 430 was withdrawn from New Zealand early in the 2000s to allow alignment of New Zealand and Australian reinforcing steel Standards.

**Grade 380** Grade 380 was a high strength, low ductility reinforcing steel used in New Zealand until ~1989. It has much poorer properties than later high strength reinforcement types.

**Non-metric** Before metrification of reinforcement occurred in 1973, New Zealand used a variety of reinforcement to British and US Standards. Most common amongst these are Grade 40 and Grade 60 bars. These have minimum yield strengths of 40 ksi and 60 ksi respectively, corresponding to metric values of 275 MPa and 414 MPa.

The details on historic reinforcing bars presented here should be considered as a brief guide only. Engineers encountering older reinforcement types in practice should conduct suitable research to ensure correct identification of reinforcement type and properties.
HARDNESS TESTING

The Engineering Advisory Group from SESOC notes “The assessment of earthquake damaged buildings must take into account the damage sustained by the building in considering its ability to resist future earthquakes.” Furthermore, “destructive testing provides the most accurate information for determining the condition of the reinforcement, provided that boundary conditions are considered, but it may not be practical for testing large numbers of locations. Non-destructive testing in conjunction with limited destructive testing (for calibration) may provide a more comprehensive coverage.”

The Leeb rebound hardness test is a method of measuring hardness of a specimen of steel, cast steel or cast iron. It is one of the four main methods for measuring hardness of materials, which include the Rockwell, Vickers and Brinell tests. Hardness of a material is a vaguely defined term that may have many meanings depending on the type of test performed and other factors (ASTM, 2012). The Leeb hardness test is of the dynamic or rebound type, which primarily depends both on the plastic and on the elastic properties of the material being tested (ASTM, 2012). The results obtained are indicative of the strength and dependent on the heat treatment of the material tested and the details of the standard method can be found in ASTM A956-12.

The Leeb test uses a portable device to impact the surface of the material being measured with a small impact body which has a spherically-shaped tip. The impact and rebound velocities of the body are measured and used to determine a value of Leeb Hardness Number. The Leeb test can be considered a non-destructive method of testing reinforcing bar surface hardness.

Measuring the surface hardness of a steel reinforcing bar which has undergone strain hardening, and comparing that value with an equivalently determined hardness value of a bar which has not undergone strain hardening, provided that appropriate boundary conditions are maintained, may be used to provide an estimation of the residual strain of that bar.

It is important that the limitations of the test method used are identified and understood. Similarly, the means of calibrating the extent of strain hardening must be robust and transparent. The ASTM Standard Test Method for Leeb Hardness Testing of Steel Products, ASTM A956-12, states:

“The Leeb hardness test is a superficial determination only measuring the condition of the surface contacted. The results generated at that location do not represent the part at any other surface location and yield no information about the material at subsurface locations.”

This statement, whilst obviously highlighting the need for tests to be undertaken at appropriate locations, emphasises the complexity involved in correlating the surface hardness with the reinforcement deterioration due to cyclic loading, and correspondingly, with the expected future performance of the overall building. This is particularly important considering the different types of reinforcement available currently and historically in New Zealand, and why an understanding of the reinforcement characteristics and manufacturing process is valuable.

Furthermore, the geometry, support conditions and surface finish of the reinforcing bar, can change the result of the measured hardness value, and must be carefully replicated in order to provide sufficient calibration between strain hardened bars in a damaged building and non-strained bars. ASTM A956-12 provides further detail on these limitations.
Finally, it is worth noting that strains caused by seismic excitation are cyclic in nature, whereas methods to correlate surface hardness with accumulated strain are monotonic. Gardiner et al. (2013) note that for structural steel elements “an assessment based on monotonic strain reserve capacity is conservative when applied to earthquake damage.”

**IMPACT OF STRAIN HARDENING**

It is important that there is a means of correlating the observed damage in a building to the expected future performance. The EAG notes that “strain hardening in itself is not a problem and in fact is required in order to force appropriate plastic hinge development.”

Bai and Au (2009) indicate that the effects of strain hardening on building performance depend on a number of parameters, including concrete strength, confining stress, reinforcement ratio, reinforcement content, and tensile-strength-to-yield-stress ratio.

The cracking patterns observed from a damage assessment are also important in assessing the implications of the strain hardening. If there is sufficient reinforcement content at the location where primary cracking (in a beam) initiates, then theoretically, additional primary and secondary cracks will form away from the initial crack, increasing the plastic hinge length.

If the cracking is concentrated at one location, generally the strain in the reinforcement will also be concentrated at that location. This can happen for a number of reasons.

- Large concentrated cracks are much more likely to be a result of insufficient quantity, not poor quality of reinforcement. Insufficient reinforcement content can cause the development of concentrated cracking (and this is the reason for having specified minimum reinforcing).

- Higher than expected bond strength between the steel reinforcing and the concrete can lead to lower strain penetration lengths, and hence more concentrated strain at a crack.

- Higher than expected concrete tensile strength due to dynamic excitations, and concrete strength gain with time can cause the concrete modulus of rupture to be higher than designed for (although more modern mix designs have a much flatter strength gain after the specified 28 days than older mix designs).

- The EAG notes that if the moment gradient is too great then additional primary cracks cannot form, and all the deformation will concentrated at the initial primary crack.

Clearly there are a large number of considerations to be taken into account when determining the implications of strain hardened steel reinforcement.

There is no question that limited strain development leading to large cracks in concrete, as opposed to the more desirable distributed cracking, has been identified as a cause for concern in reinforced concrete buildings. If there is significant strain hardening over short strain penetration lengths, and it is not possible to demonstrate that future cracking will occur elsewhere, it is considered that a reduction in assessed ductility capacity is appropriate.

Repair mechanisms likewise are not a straightforward matter. In theory, where some concentrated cracking has occurred, as long as that crack can be opened up, and filled with good quality, high strength epoxy, then any subsequent cracks should be initiated elsewhere, in a non-strain hardened region of the bar. Strain hardening correlates with some increase in
strength, and any further yielding will occur where the strength is the least (or lower). Initial yielding of a reinforcement bar is likely to occur at a location of lower strength than other areas, and can be because of a minor random imperfection. At locations of significant imperfections (defects), such as the result of spot welding or notching, highly concentrated deformations can occur leading to bar rupture or fracture. This highlights the importance of understanding the characteristics of reinforcement bars, and the limitations on handling and installation.

The SESOC Engineering Advisory Group provides more technical guidance in its document “Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury” (2013), but the following recommendation is reproduced here:

“Provided that testing shows that the reduction in strain capacity leaves no less than the required minimum strain capacity of AS/NZS4671, the building capacity may be considered to be undiminished, even though there has been some minor reduction in the total usable strain capacity of the reinforcement.”

It would however be prudent to emphasise that engineers should without exception check that reinforcement contents are sufficient to ensure distributed cracking during any assessment of existing structures (whether damaged or not). This assessment should be emphasised irrespective of the concerns regarding accumulated plastic strain.


INSURANCE CONSIDERATIONS

The EAG notes that an owner’s insurance policy may entitle the owner to a level of reinstatement that is not practically achievable without full or partial replacement of the building structure. In consideration of damage with respect to insurance, it may be considered that any strain hardening represents a reduction in total pre-existing capacity.

As noted above, some building insurance policies are worded in such a way to be used to cite strain hardening as a reason for building replacement. The view could be taken that in the plastic hinge zone (PHZ) of a beam adjacent to a column, where some (any) bar yielding has occurred beyond the yield plateau and into the strain hardening region, that region of the beam needs to be either repaired or replaced. If analysis shows that all plastic hinge zones have experienced similar strains, through rotation or other deformation, the amount of work required to replace all those sections could be considerable, particularly when taking into account the building disruption. It could be argued that to repair such sections also involves considerable cost (highlighting the necessary debate about effective and appropriate repair mechanisms). This approach could be used to demonstrate that demolishing and replacing the entire building is more cost effective than replacing only the “damaged” sections, when considering the detailed wording of the insurance policy.

Whilst the purpose of this paper is not to analyse the wording of insurance policies, it can nevertheless be expected that future insurance policies will not ignore the effects of strain hardening in concrete buildings, particularly CMRFs. Whether or not insurers wish to reassess their exposure to concrete buildings, through cover exclusions or premium increases, strain hardening is likely to be referred to, either implicitly or explicitly.
CONCRETE OPPORTUNITIES

It is worth noting that the phenomenon of steel strain hardening and a reduction in remaining plastic strain capacity is not unique to reinforced concrete buildings. Structural steel buildings in Christchurch have come under similar scrutiny (Gardiner et al., 2013), although it can also be argued that repairing steel buildings is somewhat less complicated as the damaged elements are generally more accessible than in a concrete moment resisting frame.

While as identified above, the design and construction of “traditional” CMRF structures will come under greater scrutiny, this may create opportunities to further the use of new (or less widely used) technologies, which employ damage resistant design philosophies. Largely as a result of the Christchurch earthquakes, seismic design philosophies in New Zealand are tending towards limiting non-structural damage as much as possible, instead of focussing on just designing for life-safety. The requirements to limit building damage, non-structural damage and building downtime are increasingly being given greater weighting in new structural design, particularly as a result of the cost of repairing and rebuilding many structures in Christchurch.

Anecdotal evidence from other countries with similar seismic risks, such as Japan and the west coast of the USA (particularly California), shows that buildings designed to withstand damage can attract higher yields from tenants and a reduction in building insurance premiums. Moreover, some insurers in those areas will not provide coverage for buildings which can be expected to sustain damage.

The New Zealand concrete industry has been at the forefront of such new damage resistant design concepts, with the development of both Base Isolation and PRESSS technologies originating from this country, as well as other emerging technologies, such as non-tearing joints in concrete moment resisting frames. Because of its generally high mass and high stiffness, concrete as a material is naturally well suited to such design methods.

CONCLUSIONS

- It is important to conduct a thorough assessment of, and gain a clear understanding of, the extent of damage in buildings.

- Moreover, it is important to have a thorough understanding of the material in question (steel reinforcement) and its detailed characteristics.

- Strain hardening is a complex and an on-going issue.

- There is no doubt that the insurance landscape will change, and it is important for engineers, especially in the concrete industry, to understand the factors involved.

- New damage resistant design philosophies are likely to have greater prominence in future buildings in New Zealand. Concrete as a material naturally lends itself well to such technologies.

REFERENCES

ASTM A956-12 Standard Test Method for Leeb Hardness Testing of Steel Products, ASTM International, USA


