NZS 3404:Parts 1 & 2:1997

STEEL STRUCTURES STANDARD

AMENDMENT No. 2 to Part 1

October 2007

REVISED TEXT

EXPLANATORY NOTE

This amendment brings NZS 3404:Parts 1 and 2 into line with AS/NZS 1170 *Structural design actions*, and NZS 1170.5:2004 *Structural design actions – Earthquake actions*.

APPROVAL

Amendment No. 2 was approved on 23 October 2007 by the Standards Council to be an amendment to NZS 3404:Parts 1 and 2:1997.

In all Part 1 and Part 2 sections except section 12, where the Standard specifies NZS 4203 this shall be read as AS/NZS 1170 set.

In Part 1 and Part 2 of section 12, where the Standard specifies NZS 4203 this shall be read as NZS 1170.5.

(Amendment No.2, October 2007)

Delete the terms G^* and Q^* wherever they appear in the Standard.

(Amendment No.2, October 2007)

CONTENTS (page 5)

Delete "Appendix M".

(Amendment No.2, October 2007)

1.1.4(b) (page 13)

Add to the end of this subclause:

"with regard to any reduction in mechanical properties due to the fabrication process."

(Amendment No.2, October 2007)

1.1.5.3 (page 14)

Delete the whole subclause and **substitute**:

"Use of Part 1 of this Standard for design of structures which are outside the scope of AS/NZS 1170.0 clauses 1.1 and 1.2, such as those based on special studies or those relying on other loadings documents, comprises an alternative solution and compliance with the Building Code shall be demonstrated."

(Amendment No.2, October 2007)

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1.3 DEFINITIONS (page 14)

Definition of ACTION (page 14)
Delete the words "due to effects or loads".
Add "NOTE – In AS/NZS 1170 this is defined as ACTION EFFECT."

Add a new definition after CAPACITY DESIGN (page 15)
"CAPACITY DESIGN ACTION. The design action derived from application of capacity design in accordance with 12.2.7."

Definition of DESIGN CAPACITY (page 16)
ADD "NOTE – In AS/NZS 1170 this is defined as DESIGN STRENGTH".

Definition of DESIGN ENGINEER (page 16)
Delete the definition and **substitute:**
"A person who shall be a chartered professional engineer, is registered under the CPEng of NZ Act 2002 and who is competent to design structural elements of the building under consideration to safely resist the design loads or effects likely to be imposed on the building."

Definition of LOAD (page 18)
Add "NOTE – In AS/NZS 1170 this is defined as ACTION".
Add "LOAD, DEAD. Is referred to in AS/NZS 1170 set as permanent action".
Add "LOAD, LIVE. Is referred to in AS/NZS 1170 set as imposed action".

Definition of LOADINGS STANDARD (page 18)
Delete "NZS 4203" and **substitute**:
"AS/NZS 1170 set". This is comprised of AS/NZS 1170.0, AS/NZS 1170.1, AS/NZS 1170.2, AS/NZS 1170.3, and NZS 1170.5.

Definition of NOMINAL CAPACITY (page 19)
Add "NOTE – In NZS 1170.5 this is defined as NOMINAL STRENGTH".

Definition of NZS 4203
Delete the definition of NZS 4203.

Definition of OWNER (page 19)
Delete "… Building Act 1991" and **substitute:**
"… Building Act 2004".

Definition of PRIMARY SEISMIC-RESISTING ELEMENT (page 20)
Delete the definition and **substitute**:
"An element or member of a seismic-resisting system chosen and designed to be part of the main energy dissipating mechanism."

Definition of SECOND-ORDER ANALYSIS. (page 21)
Delete the definition and **substitute:**
"A (frame) analysis (elastic, unless started otherwise) in which the derivation of member actions is based in the deformed shape of the structure and also includes, when required, the effects of reduction in elastic stiffness of the members due to axial compressive forces".

Definition of SECONDARY SEISMIC-RESISTING ELEMENT (page 21)
Change "element" **to** "element or member", twice in line 1.

Definition of STRUCTURAL PERFORMANCE FACTOR (page 22)
Delete "NZS 4203" and **substitute** "NZS 1170.5".

(Amendment No.2, October 2007)

1.4 NOTATION (page 23)

Notation C, C(T) (page 25) **delete** the notation and **substitute:**

"C_d (T_1) = horizontal design action co-efficient, as determined from NZS 1170.5 for use in section 12.

Notation c_h (page 25) **delete.**

Notation G (page 28) **delete** "nominal".

Notation Q (page 34) **delete** "nominal" and **add** "= design transverse force".

Notation Q^* (page 34) **delete.**

Add to page 27 after f'_c

"f'_{cos} = the long term increase in concrete stress in the slab above the nominal 28 day strength, taken as 10 MPa"

Add to page 33 after N^*

"N^*_{oc} = the capacity design derived design axial compression force on the column when capacity design is used and the column is a secondary element to 12.2.7.1 and 12.2.7.4"

(Amendment No.2, October 2007)

1.5.1 (a) (page 42)

Delete item (a) and **substitute:**

(a) A special study, in accordance with subclause 1.4.22 of AS/NZS 1170.0 or clause 1.4 of NZS 1170.5; or

1.5.1 (page 42)

In the last paragraph **delete** "Building Act 1991" and **substitute** "Building Act 2004".

(Amendment No.2, October 2007)

1.6.2 (page 42)

Add item (k)

"(k) The fire protection requirements, if applicable".

(Amendment No.2, October 2007)

1.6.3.2 (page 43)

Delete NOTE (1) and **substitute:**

"(1) A construction reviewer might be a Chartered Professional Engineer with suitable experience."

(Amendment No.2, October 2007)

2.2 STRUCTURAL STEEL (page 45)

2.2.1 Specification, item (b)

Delete "BS 6363 Specification for welded cold formed steel structural hollow sections", and **substitute** "BS EN 10219 Cold formed welded structural hollow sections of non-alloy and fine grain steels. Part 2 Tolerances, dimensions and sectional properties".

Delete "BS 7613 Specification for hot rolled quenched and tempered weldable structural steel plates."

Delete "BS 7668 Specification for weldable structural steels. Hot finished structural hollow sections in weather resistant steels" and **substitute** "BS 7668 Weldable structural steels. Hot finished structural hollow sections in weather resistant steels. Specification"

Delete "BS EN 10025 Hot rolled products of non-alloy structural steel. Technical delivery conditions".

Add "BS EN 10025 Hot rolled products of structural steels
- Part 1 General delivery conditions
- Part 2 Technical delivery conditions for non-alloy structural steels
- Part 3 Technical delivery conditions for long products
- Part 4 Technical delivery conditions for the thermomechanical rolled weldable fine grain steels
- Part 5 Technical delivery conditions for structural steels with improved atmospheric corrosion resistance
- Part 6 Technical delivery conditions for plates and wide flats of high yield strength structural steels in the quenched and tempered condition".

Delete "BS EN 10113 Hot rolled products in weldable fine grain structural steels
- Part 1 General Delivery conditions
- Part 2 Delivery conditions for normalized/normalized rolled steels
- Part 3 Delivery conditions for thermomechanical rolled steels"

Delete BS EN 10155 Structural steels with improved atmospheric corrosion resistance. Technical delivery conditions

Add to 2.2.1(c) (page 46) "JIS G 3136 Rolled steel for building structure"

(Amendment No.2, October 2007)

2.2.1 Specification (page 45)

In line 2 **delete "**(a), (b), (c) and (d)" and **substitute** "(a), (b), (c) or (d)".

2.2.1(a) (page 45)

In line one **delete** the words "New Zealand or Australian steels shall comply with the requirements of the appropriate following Australian Standards:" and **subsititute** "Australian or Joint Australian/New Zealand Standards".

2.2.1(b) (page 45)

Delete the words "British steels shall comply with the requirements of the appropriate following British Standards" and **substitute** "British Standards".

2.2.1(c) (page 46)

Delete the words "Japanese steels shall comply with the requirements of the appropriate following Japanese Standards" and **substitute** "Japanese Standards".

2.2.1(d) (page 46)

Add new sentence "Such an approval is outside the scope of this Standard as a Verification Method for the NZ Building Code."

(Amendment No.2, October 2007)

2.2.3 Unidentified steel (page 46)

Delete the last paragraph of subclause 2.2.3 and **substitute** with the following two paragraphs:

Unidentified steel shall not be used as elements in the seismic-resisting system. However when evaluating the performance of existing structures, unidentified steel may be used if shown by tests to meet the material requirements of 12.4.

Unidentified steel shall not be used in members of an associated structural system which are subject to inelastic demand (see subclause 12.3.4.2) or in members which are subject to moment redistribution, ***unless*** it is shown by tests that the steel complies with the elongation requirements of 4.6.2(b)(iii).

(Amendment No.2, October 2007)

2.3.1 Steel bolts, nuts and washers (page 46)

Delete references to AS/NZS 1110 ISO metric precision hexagon bolts and screws, AS/NZS 1111 ISO metric hexagon commercial bolts and screws and AS/NZS 1112 ISO metric hexagon nuts, including thin nuts, slotted nuts and castle nuts and **substitute:**

AS 1110	ISO metric hexagon bolts and screws Product grades A and B
AS 1111	ISO metric hexagon bolts and screws Product grades C
AS 1112:Part 1	ISO metric hexagon nuts – Style 1 Product grades A and B

(Amendment No.2, October 2007)

2.5 CONCRETE (page 47)

Delete the whole clause and **substitute**:

"Unless otherwise required by this Standard, all structural and fire protective concrete used in association with structural steel shall comply with NZS 3104 and NZS 3109."

(Amendment No.2, October 2007)

2.6 MATERIAL SELECTION TO SUPPRESS BRITTLE FRACTURE (page 47)

2.6.1 Methods

Add a new paragraph:

“For steels used in seismic-resisting systems, refer to table 12.4.”

(Amendment No.2, October 2007)

2.6.4.5.2 (page 51)

Delete the whole subclause and **substitute**:

“For welds subject to earthquake loads or effects (see 9.7.1.4.3(b)), the following shall apply:

(a) The welding consumables shall have a Ships Classification Societies Grade 3 approval as shown in table 4.6.1(A) of AS/NZS 1554.1;2004, as required for Steel Type 3 for Grade 300 steel, Steel Type 6 for Grade 350 steel and Steel Type 7C for Grade 450 steel.

(b) The heat input in the deposited weld metal shall not exceed 2.5 kJ/mm run.”

(Amendment No.2, October 2007)

2.6.5 Fracture assessment (page 51)

Delete NOTE and **substitute**:

“NOTE – For methods of fracture assessment, see BS 7910 Guide on methods for assessing the acceptability of flaws in metallic structures.”

Add a second paragraph:

“An assessment made under this subclause is outside the scope of this Standard as a Verification Method for the NZ Building Code”.

(Amendment No.2, October 2007)

Table 2.6.4.4 Steel type relationship to steel grade (page 51)

Delete the whole table (including the notes, (There are no notes in the substituted table) and **substitute:**

Table 2.6.4.4 – Steel type relationship to steel grade
(For steels from 2.2.1)

Steel Type	Steel Grade					
	AS 1163	**AS 1594**	**AS/NZS 3678 AS/NZS 3679.2**	**AS 3679.1**	**BS EN 10025**	**JIS G 3106 JIS G 3136**
1	C250	HA200 HA250, HU250 HA300, HU300 HA300/1	200 250 300	250 300	S275 S275JR	SM 400A SN 400A
2	C250L0	-	-	250L0 300L0	S275J0	SM 400B SN 400B
3	-	-	250L15 300L15	250L15 300L15	S275J2G3/ S275J2G4	SM 400C
4	C350	HA350 HA400	350 WR350	350	S355 S355JR	SM 490YA
5	C350L0	-	WR350L0	350L0	S355JO	SM 490YB SM 520B SN 490B
6	-	-	350L15 400L15	350L15	S355J2G3/ S355J2G4	SM 520C
7A	C450	-	-	-	-	-
7B	C450L0	-	-	-	-	-
7C	-	-	450L15	-	-	-

(Amendment No.2, October 2007)

3.2.1 Loads (page 53)

In item (b) **delete** "NZS/BS 2573 or".
In item (c) **delete** "AS 1657" and **substitute** "NZS/AS 1657".
In item (d) **delete** "NZS 4332P" and **substitute** "NZS 4332".

Add a new NOTE (3):

(3) Only loads determined from (a) shall be used when this Standard is used as a Verification Method for Clause B1 of the New Zealand Building Code.

(Amendment No.2, October 2007)

3.2.4 Notional horizontal loads (page 54)

Delete the existing subclause 3.2.4 and **substitute:**

"**3.2.4 Structural robustness**

3.2.4.1 Minimum lateral resistance of the completed structure

Completed structures shall meet the minimum lateral resistance requirements of AS/NZS 1170.0 subclause 6.2.2.

3.2.4.2 Construction stage

Structural robustness during construction shall comply with the requirements of AS 3828. "

(Amendment No.2, October 2007)

3.4.6 Corrosion protection (page 57)

Add to the end of the NOTE:
", although this is not cited as a Verification Method for Clause B2 of the NZBC."

(Amendment No.2, October 2007)

3.7.2 (page 58)

Delete this subclause.

(Amendment No.2, October 2007)

4.3.3 Arrangements of live loads for buildings (page 61)

Delete the whole subclause and **substitute:**
"Arrangement of live (imposed) loads shall comply with AS/NZS 1170.1 clause 3.3."

(Amendment No.2, October 2007)

4.5.7.2.1 *Connection design moment* (page 71)

In line 1 of item (a) **delete** "($\phi_{om} M_s$)" and **substitute** "(0.9 $\phi_{om} M_s$)".

(Amendment No.2, October 2007)

4.7.2 Plastic hinge rotation limits (page 74)

Delete tables 4.7(1) to 4.7 (4) and their notes and **substitute**:

"(a) For members with negligible axial force ($N^* \leq 0.15\ \phi N_s$)

Table 4.7(1) – Limiting plastic hinge rotation (Radians x 10^{-3}) for load combinations

Category of member	Including earthquake loads or effects	Not including earthquake loads or effects
1	40	60
2	40	60
3	30	45

(b) For low axially loaded members ($0.15\ \phi N_s < N^* \leq 0.3\ \phi N_s$)

Table 4.7(2) – Limiting plastic hinge rotation (Radians x 10^{-3}) for load combinations

Category of member	Including earthquake loads or effects	Not including earthquake loads or effects
1	30	45
2	30	45
3	20	30

(c) For moderately axially loaded members ($0.3\ \phi N_s < N^* \leq 0.5\ \phi N_s$)

Table 4.7(3) – Limiting plastic hinge rotation (Radians x 10^{-3}) for load combinations

Category of member	Including earthquake loads or effects	Not including earthquake loads or effects
1	13	20
2	13	20
3	10	15

(d) For highly axially loaded members ($0.5\ \phi N_s < N^* \leq 0.8\ \phi N_s$)

Table 4.7(4) – Limiting plastic hinge rotation (Radians x 10^{-3}) for load combinations

Category of member	Including earthquake loads or effects	Not including earthquake loads or effects
1	8	15
2	8	15
3	5	10

Notes for tables 4.7(1) to 4.7(4)

(1) The category of member applies to 4.5.4.1, 4.6 or 12.3 as appropriate.
(2) The plastic hinge rotation as determined from structural analysis (θ_p^*) must be based on a plastic hinge length not exceeding the member depth.
(3) Where the analysis models the plastic hinge rotation as a spring, this shall be positioned at the centre of the plastic hinge length determined from NOTE (2), except for members with plastic hinges only at the member ends, the plastic hinge shall be positioned at the member ends.
(4) When the design axial force (N^*) includes earthquake loads, N^* shall be the mean axial force over one cycle of loading.
(5) Plastic rotation limits for active links in eccentrically braced frames are given in 12.11.3.3."

(Amendment No.2, October 2007)

4.9.1 General (page 80)

Add to the end of the subclause:

"NOTE – $\lambda_c \geq 3.5$ is required when applying 4.9.2. "

(Amendment No.2, October 2007)

8.4.3.2.1 (page 151)

Delete the two existing equations and **substitute**:

$$“\frac{N^*}{\phi N_s} \leq \left\{\frac{0.263(\beta_m + 1)^{0.88}}{e^{(0.19/(\beta_m+1))}}\right\}^{\lambda_{EYC}}$$

where

$$\lambda_{EYC} = \sqrt{\frac{N_s}{N_{oL}}}$$

All other variables are as defined on page 151 of the Standard. "

(Amendment No.2, October 2007)

8.4.3.2.2 (page 151)

Delete the whole subclause and **substitute**:

"**8.4.3.2.2**

A member subject to design axial compression force which does not satisfy 8.4.3.2.1 shall not contain plastic hinges, although it shall be permissible to design the member as an elastic member in a plastically analysed structure to satisfy the requirements of 8.4.2. "

(Amendment No.2, October 2007)

9.1.3 Design of connections (page 157)

Add the following as a new subclause **9.1.3.5.**

9.1.3.5

Gusset (and other) unstiffened plates designed to transfer compression forces shall be subject to rational analysis to :

(a) Satisfy the strength and stability requirements of section 8;

(b) Incorporate allowances for eccentric load effects;

in accordance with 9.1.5.1.

(Amendment No.2, October 2007)

9.1.4.1 Minimum design actions on connections not subject to earthquake loads or effects (page 158)

In item (b)(v) **delete** the first paragraph of the subclause and **substitute** as below. The second and third paragraphs remain.

"(v) Splices in members subject to axial compression for ends prepared for full contact in accordance with 14.4.4.2, shall be permissible to carry compressive actions by bearing on contact surfaces. When members are prepared for full contact to bear at splices, there shall be sufficient fasteners to hold all parts in place. The fasteners shall be sufficient to transmit a force of 0.15 times the member design strength in axial compression. Fasteners shall also be sufficient to transmit a minimum of 0.15 times the member flexural design strength in tension friction mode."

Add the following new subclause to the end of 9.1.4.1 (page 159):

(c) Members shall be connected in such a way to meet the robustness requirements of AS/NZS 1170.0 subclause 6.2.3, with the design action applied along the longitudinal axis of the supported member.

(Amendment No.2, October 2007)

9.3.1 Bolts and bolting category (page 163)

In line one of **9.3.1.2 delete** references to AS/NZS 1110 and AS/NZS 1111 and **substitute** "AS 1110 and AS 1111".

(Amendment No.2, October 2007)

10.2.2 (page 188)

In 10.2.2 **delete** reference to NZS/BS 2573, Rules for the design of cranes, parts 1 and 2.

(Amendment No.2, October 2007)

11.7.3.2 Limitations and conditions on use of regression analysis (page 210)

In the NOTE to **11.7.3.2.1**(a) **replace** the second sentence with the following two sentences: "They may be tested and assessed in accordance with section 5 of the document *Fire Protection for Structural Steel in Buildings,* (4th edition). In that publication, intumescent paints are referred to as reactive fire protection systems*".*

In 11.7.3.2.2 **replace** the third sentence with:
"Section 3 of the document *Fire Protection for Structural Steel in Buildings* (4th edition), provides details of how to undertake this interpolation as part of an assessment of the performance of passive fire protection products"

(Amendment No.2, October 2007)

12.2.2.1 Structural performance factor (page 215)

Delete this subclause and **substitute**:

"**12.2.2.1 Structural performance factor values**

The structural performance factor, S_p, shall be taken as equal to:

(a) For the serviceability limit state: .... $S_p = 0.7$

(b) For the ultimate limit state:

  (i) For category 1 structures .... $S_p = 0.7$

  (ii) For category 2 structures .... $S_p = 0.7$

  (iii) For category 3 structures .... $S_p = 0.9$

  (iv) For category 4 structures .... $S_p = 0.9$

For the ultimate limit state for category 3 or 4 systems, if all the elements of the system meet the material, section geometry, member restraint and connection requirements of section 12 for category 2 members, then $S_p = 0.7$. "

(Amendment No.2, October 2007)

12.2.3.1 Categories of ductility demand (page 215)

There is no change to the opening sentence or to (1) and (2).

Delete subclause (3) and **substitute**:

"(3) *Nominally ductile systems* (Category 3 systems)
These are to be capable of sustaining structural displacement ductility demands sufficient to yield the flanges of primary seismic-resisting members or elements under the design level ultimate limit state earthquake loads or effects and to resist collapse under a maximum considered earthquake as directed by the Loadings Standard. "

Delete subclause (4) and **substitute**:

"(4) *Elastic systems* (Category 4 systems)
These are expected to respond with minimal structural displacement ductility demand under the design level ultimate earthquake loads or effects and to resist collapse under a maximum considered earthquake as directed by the Loadings Standard. Elastic systems are not brittle systems: brittle systems are outside the scope of this Standard. "

(Amendment No.2, October 2007)

12.2.3.3 Application of structural classifications (page 216)

Delete the whole subclause from (1) to the end, and **substitute:**

"(1) *Category 1 and 2 systems*
Capacity design procedures are required and the effects of concurrent action on columns which form part of a two-way seismic-resisting system shall be considered in accordance with 12.8.4(a) or (b).

(2) *Category 3 systems*
Capacity design procedures are required for category 3 structures which do not comply with table 12.2.6 Case Number 4 or as required by 12.11.1.1 or 12.12.5.3 and design for concurrent action is required by 12.8.4(b) or (c).

(3) *Category 4 systems*
Capacity design procedures are not required. Design for concurrent action is required by 12.8.4 (c)."

For systems including columns subject to concurrent actions (see 12.8.4) or for dual systems (see 12.13) the category of each system shall not differ numerically by more than one.

(Amendment No.2, October 2007)

12.2.4 Structural displacement ductility demands (page 216)

Delete the first sentence and **substitute:**
Structural displacement ductility demands on the 4 categories of seismic-resisting systems for the ultimate limit state are specified in table 12.2.4.

Add the words "**for the ultimate limit state**" to the end of the title for table 12.2.4,

Add the following new paragraph to this subclause:
For the serviceablility limit states defined in NZS 1170.5, the structural displacement ductility factor shall be 1.0 ≤ μ ≤ 1.25 for SLS1 and 1.0 ≤ μ ≤ 2 for SLS2.

(Amendment No.2, October 2007)

12.2.6 Relationship between structure category and member category (page 217)

Delete the whole subclause including table 12.2.6 and **substitute**:
This subclause applies to seismic-resisting systems in accordance with 12.3.2 and 12.3.3 and where appropriate, to associated structural systems in accordance with 12.3.4.

For systems outside the scope of table 12.2.6 the member category for each member of the seismic-resisting system shall be determined from matching the plastic hinge demand determined from analysis to 12.3.2.1 to the plastic hinge rotation limits of 4.7.2.

Other than an EBF active link (refer 12.11.3.3.1), plastic hinge rotation limits of 4.7.2 must also be checked for the members of any seismic-resisting system in which the clear length L of any primary member is $\leq 3\ M_s/V_w$.

Table 12.2.6 specifies the relationship between structural system and member category for the following applications:

(a) For seismic-resisting systems in which capacity design to prevent soft storey mechanisms from forming has been undertaken, except for category 1 to 4 concentrically braced frame systems which are covered by table 12.12.5.1.
(b) For category 3 seismic-resisting systems not exceeding the critical height (see 1.3) in which capacity design is not undertaken and for which the building is not irregular when assessed to the requirements of the Loadings Standard.
(c) For category 4 seismic-resisting systems not exceeding the critical height (see 1.3) and for which the building is not irregular when assessed to the requirements of the Loadings Standard.

(Amendment No.2, October 2007)

Table 12.2.6 – Relationship between structure category and member category (page 218)

Delete table 12.2.6 and the NOTES and **substitute**:

"**Table 12.2.6 – Relationship between structure category and member category**
(Except as specified in limits on application (a) to (c) above and Notes 5, 6)

Case number	**Structural ductility category**	**Capacity design to prevent soft storey mechanism undertaken**	**Type of member of structural system** (See Note 1)	**Minimum member ductility category**
1	1	Yes	Primary Secondary	1 2 (3) (See Note 2)
2	2	Yes	Primary Secondary	2 2 (3) (See Note 2)
3	3	Yes	Primary Secondary	3 3
4	3	No	Columns All other members	2 (See Notes 1, 3) 3
5	4	No	Columns All other members	3 (See Note 4) 4 (See Note 4)

NOTE –
(1) When a capacity design to prevent a soft storey mechanism is not undertaken, all members of the structural system are classified as primary (seismic-resisting) members. Because a column sidesway mechanism is not suppressed, more stringent minimum member ductility category requirements apply to the columns than to the other members of a structural system for which capacity design is not undertaken.
(2) The unbracketed value applies to secondary structural members which yield as the structure deforms laterally to obtain the desired mechanism (e.g. at the bottom of the columns of the storey immediately above the base). The bracketed value applies to other secondary members (e.g. in columns which may undergo lower levels of yielding above the base due to dynamic effects).
(3) Case number 4 is restricted by the limit on application (b) above. For single storey buildings, columns can be category 3.
(4) Case number 5 is restricted by the limit on application (c) above. For single storey buildings, columns can be category 4.
(5) For concentrically braced frame seismic-resisting systems, the relationship between structure category and member category is given in table 12.12.5.1.
(6) For eccentrically braced frames, see 12.11.3.2 for application of this table."

(Amendment No.2, October 2007)

12.2.8 Overstrength (page 219)

Add the following NOTE (6) to tables 12.2.8:

"(6) For moment-resisting frames and for long link D braced EBF seismic-resisting systems comprising beams supporting a concrete slab, refer to 12.10.2.4 and 12.11.7.2."

(Amendment No.2, October 2007)

12.2.9.2 Influence of damping values on the ultimate limit state design seismic load for category 3 and 4 structural systems (page 220)

In the second sentence, **delete**:
"Where a value of damping other than 5 % is applicable from 12.2.9.1," and **substitute:**
"Where a value of damping greater than 5 % is applicable from 12.2.9.1, and the zone factor Z, from NZS 1170.5 is greater than 0.160 …".

(Amendment No.2, October 2007)

12.3.2.1.5 (page 222)

Add a new sentence:

"For category 1 and 2 concentrically braced frames, the lateral deflection shall be increased as required by 12.12.5.2 (h) ".

(Amendment No.2, October 2007)

12.3.3.4 Maximum design actions for members of seismic-resisting systems (page 223)

Delete paragraph one and **substitute**:

The members of seismic-resisting systems need not be designed to transmit design actions greater than (a) or (b) below.

Replace subclause (a) with the following:

(a) Those generated by nominally ductile seismic response in conjunction with the appropriate design gravity loads for category 1 seismic-resisting systems or for category 2 seismic-resisting systems with $\mu_{act} \geq 1.8$. The value of S_p used shall be 0.7.

Replace subclause (b) with the following:

(b) Those generated by elastic seismic response in conjunction with the appropriate design gravity loads for category 3 or 4 seismic-resisting systems or for category 2 seismic-resisting systems with $\mu_{act} < 1.8$. The value of S_p used shall be 0.7 for category 2 systems, 0.9 for category 3 seismic-resisting systems and 0.9 for category 4 seismic-resisting systems.

(Amendment No.2, October 2007)

12.4.1 Material requirements for category 1, 2 and 3 members (page 226)

Delete "for category 1,2 and 3 members" from the heading.

(Amendment No.2, October 2007)

12.4.1.1 (page 226)

Delete the whole subclause including table 12.4 and NOTES and **substitute:**

12.4.1.1
"Steels in seismic-resisting systems must meet the criteria in table 12.4:

Table 12.4 – Material requirements

Item		Category 1, 2 and 3 members	Category 4 members
1	Maximum specified grade reference yield stress (See Note 1)	360 MPa	450 MPa
2	Minimum % total actual elongation (See Note 2, 3)	25	15
3	Maximum actual yield ratio (f_y/f_u) (See Note 2)	0.80	0.90
4	Maximum actual (See Note 2) yield stress	≤ 1.33f_y (See Note 4)	-
5	Minimum Charpy V-Notch impact energy (See Notes 3, 5, 6)	70J @ 0 °C Average of three tests 50J @ 0 °C Individual test	No special earthquake provisions required

NOTE –
(1) The limits in item 1 are based on a grade reference steel thickness of 12 < t ≤ 20 mm from the appropriate materials supply Standard from 2.2.1.
(2) For items 2, 3 and 4, the mechanical properties are those recorded on the certified mill test report or test certificate.
(3) Tensile and Charpy V-Notch testing shall be completed and assessed for compliance in accordance with the provisions for selection, position and orientation, preparation for testing and testing procedures found in AS/NZS 3679.1 for hot rolled steel sections, AS/NZ 3678 for plate used in welded steel sections and AS 1163 for structural steel hollow sections.
(4) f_y is the specified yield stress from 2.1.1.
(5) Charpy V-Notch testing is only required for sections greater than 12 mm thick.
(6) These impact requirements are for steel in environments where the basic service temperature (see 2.6.3) is ≥ 5 °C.

(Amendment No.2, October 2007)

12.4.2 (page 226)

Delete the subclause.

(Amendment No.2, October 2007)

12.8.3.1 Limitations on axial force (pages 233 and 234)

Delete the first sentence and **substitute** with the following two sentences:

The ratio of design axial force, N^*, to design section capacity, ϕN_s,(refer to 6.2) shall not exceed the values given in (a) and (b) below.

The ratio of design axial force generated by gravity loading alone, N_g^*, to design section capacity, ϕN_s, shall not exceed the value given in (c) below.

Delete the existing Equation 12.8.3 1 in item (b) and **substitute**:
Where capacity design is not undertaken:

$$\frac{N^*}{\phi N_s} \le \left\{ \frac{0.263(\beta_m + 1)^{0.88}}{e^{(0.19/(\beta_m+1))}} \right\}^{\lambda_{EYC}} \quad \text{(Eq.12.8.3.1(1))}$$

Where capacity design is undertaken and the column is the secondary element to 12.2.7.1:

$$\frac{N^*_{oc}}{\phi N_s} \le \left\{ \frac{0.263(\beta_m + 1)^{0.88}}{e^{(0.19/(\beta_m+1))}} \right\}^{\lambda_{EYC}} \quad \text{(Eq.12.8.3.1(2))}$$

where

$$\lambda_{EYC} = \sqrt{\frac{N_s}{N_{oL}}}$$

N^*_{oc} = the capacity design derived design axial compression force on the column when the column is a secondary element to 12.2.7.1

N^* = the design axial compression force

All other variables are as defined in the Standard.

The scope of application of the equation is as defined in the Standard.

Retain the last paragraph in 12.8.3.1 item (b)

(Amendment No.2, October 2007)

12.8.4 Concurrent action on columns (page 235)

Delete the whole subclause on pages 235 and 236 and **substitute**:

"For columns (and their foundations) which are part of a two-way seismic-resisting system, concurrent action shall be considered (allowing for the sign of the action when a 3 dimensional analysis is undertaken) in accordance with (a) to (c) as applicable:

(a) *Both seismic-resisting systems are category 1 or category 2*
- (i) The seismic-resisting systems shall be designed, using capacity design, for seismic forces applied separately along each principal direction; then
- (ii) The capacity design derived design actions from (a)(i) shall be considered acting concurrently on the column; and
- (iii) Column design shall be to the requirements of sections 5-8 and 13; and
- (iv) The design strength of the column shall be not less than the concurrently acting design actions (S^*).

(b) *One seismic-resisting system is category 2, the other is category 3*
- (i) The seismic-resisting systems shall be designed for the seismic forces applied separately along each principal direction, using capacity design derived actions for the category 2 system and capacity design derived actions or design actions, as appropriate, for the category 3 system; and
- (ii) The category 3 concurrent design actions resulting from 100 % of the earthquake forces acting on the category 3 system shall be determined; then
- (iii) 100 % of the capacity design derived design actions from the category 2 system in conjunction with 30 % the category 3 concurrent design actions from (ii) above shall be considered acting concurrently on the column; and
- (iv) Column design shall be to the requirements of sections 5-8 and 13; and
- (v) The design strength of the column shall be not less than the concurrently acting design actions (S^*).

(c) *Both seismic-resisting systems are either category 3 or category 4*
- (i) The seismic-resisting systems shall be designed for seismic forces applied separately along each principal direction, using capacity design where appropriate; then
- (ii) The column shall be designed for concurrent design actions generated by application of the specified seismic forces (and gravity loads) from the Loadings Standard, with the seismic forces acting on the structure in the direction that produces the most unfavourable effect in that column member; and
- (iii) Column design shall be to the requirements of sections 5-8 and 13; and
- (iv) The design strength of the column shall be not less than the concurrently acting design actions (S^*).

The category of system for application of the concurrent actions shall be assessed to μ_{act} when this is calculated; otherwise assessed to μ."

(Amendment No.2, October 2007)

12.9.1.2.2 (page 237)

In the first sentence of 12.9.1.2.2, **delete** the words "subject to these actions not being less than the minimum design actions specified by 12.9.2".

(Amendment No.2, October 2007)

12.9.1.2.2 (2) (page 238)

Delete the whole of 12.9.1.2.2(2) and **substitute**:

(2) *Connectors and connection components between elements of primary category 4 members*

(a) The design actions shall be as determined from structural analysis in accordance with section 4 and the appropriate provisions of section 12, using S_p = 1.0.

(b) The design capacity of the connectors or connection components shall be used to resist the design actions.

(Amendment No.2, October 2007)

12.9.1.2.2 (4) (page 238)

Delete the whole of 12.9.1.2.2(4) and **substitute**:

“(4) *Upper limit design actions on connections in seismic-resisting structural systems*
Connections need not be designed to transmit actions greater than (a) or (b) below, and (c) where applicable:

(a) Those generated by nominally ductile seismic response in conjunction with the appropriate design gravity loads for category 1 seismic-resisting systems or for category 2 seismic-resisting systems with $\mu_{act} \geq 1.8$. The value of S_p used shall be 0.7.

(b) Those generated by elastic seismic response in conjunction with the appropriate design gravity loads for category 3 seismic-resisting systems or for category 2 seismic-resisting systems with $\mu_{act} < 1.8$. The value of S_p used shall be 0.7 for category 2 systems and 0.9 for category 3 seismic-resisting systems.

NOTE – Category 4 seismic-resisting system design actions given by 12.9.1.2.2(2) are upper limit actions. ”

Add a new subclause (c) as follows:

(c) For connections in category 1, 2 or 3 seismic-resisting systems incorporating incomplete penetration butt welds, fillet welds, bolts and pins, when the seismic-resisting system requires capacity design in accordance with 12.2.6 and the capacity design derived design actions on any connection is limited by (a) or (b) above, the connection shall be designed to resist 1.25 times the actions generated by the design capacity of the primary member or members to which it is attached.”

(Amendment No.2, October 2007)

12.9.2 Minimum design actions on connections subject to earthquake loads or effects (page 239)

Add the following two new paragraphs after the existing sentence:

“The minimum design actions required for all connections shall be given by the lesser of 12.9.1.2.2(4) or the relevant paragraph below.”

In addition, members shall be connected in such a way to meet the robustness requirements of AS/NZS 1170.0 subclause 6.2.3, with the design action applied along the longitudinal axis of the supported member. ”

(Amendment No.2, October 2007)

12.9.5.2 (b) (ii) (2) (page 243)

Add the following after subclause (2):

"Where required by 12.10.2.3, for category 1, 2 and 3 MRFs of rigid construction (see 4.2.2.1), incorporating welded beam to column connections, the design actions on the panel zone shall include the strength increase due to the slab through multiplying the term C_2 by the factor :

$(1.0 + 0.54\ t_{ef}/d_b)$

where

d_b = the steel beam depth (if the beams are a different size, use the larger value)

t_{ef} = the thickness of the concrete rib in direct contact with the column

= t_o when the concrete rib is continuous along the beam (e.g. when the decking is parallel to the beam), otherwise

= t (from 13.1.2.5.1)."

(Amendment No.2, October 2007)

12.9.5.4.4 (page 246)

Delete the existing subclause.

(Amendment No.2, October 2007)

12.9.7 Design of gusset plates (page 247)

Add the following subclause:

"**12.9.7.4 Allowance for beam-column joint opening**
Where a gusset plate is welded or bolted into both a beam and a column member, the connections between the gusset plate and the supporting members shall be designed and detailed to accommodate opening of the joint angle under inelastic seismic action. "

(Amendment No.2, October 2007)

12.10.2 Design procedure (page 248)

Retain the first sentence of the existing 12.10.2, and **add** the following sentence:

"The capacity design procedure for moment-resisting steel frames must meet the following requirements:"

Then **add** the following new subclauses:

"**12.10.2.1**
The beams are the primary members, the columns and connections are the secondary members and elements.

12.10.2.2
Unidirectional beam hinging shall be suppressed.

12.10.2.3
The columns and connections shall be designed to resist the overstrength actions from the beams. These are developed at the supported end of the beams.

For beams in MRFs of rigid construction (see 4.2.2.1) in which the slab is isolated from the column, the overstrength actions from each beam developed at the column face for design of the column member for combined bending and axial force shall be given as $\phi_{oms}M_s$, where ϕ_{oms} is given by table 12.2.8.(1) and M_s is the nominal section moment capacity of the beam from 5.2.

(Amendment No.2, October 2007)

For category 1, 2 and 3 MRFs of rigid construction in which the slab is not isolated from the column, the influence of the slab shall be included in the overstrength actions for design of the column member for combined bending and axial force in accordance with 12.10.2.4.

12.10.2 4
Where required by 12.10.2.3, for category 1, 2 and 3 for MRFs of rigid construction (see 4.2.2.1), or by 12.11.7.2 for D-braced EBFs with long active links, the overstrength moment from the composite beams at the column face, $M°$, shall include the strength increase due to the slab as follows.

$$M° = \Sigma M°_{i} + N_{slab}(d_b/2+t_o-t_{ef}/2)$$

where
$\Sigma M°_{i}$ is the sum of the overstrength moments i considering N_{slab} as:

$$\Sigma M°_{i} = \min\{1.18 \times (1 - N_{slab}/\Sigma(A_g f_y)_i) \times \Sigma M°_{b,I}, \Sigma M°_{b,i}\}$$

and
N_{slab} is the axial force generated by the slab, given by:

$$N_{slab} = \min\{1.3 t_{ef} b_{sef}(f'_c + f'_{cos}); \Sigma(A_g F_y)_i\}$$

where

A_g	=	area of steel beam framing into the column
b_{sef}	=	the width of compression action against the supporting column
	=	b_{fc} when the column is an I section and the beam frames into the column flange
	=	d_c when the column is an I section and the beam frames into the column web
	=	$0.9d_c$ when the column is a concrete filled circular steel tube
	=	the width of column perpendicular to the incoming beam when the column is a concrete filled square or rectangular hollow section
d_b	=	the steel beam depth (if the beams are different size, use the larger value)
f'_{cos}	=	the long term increase in concrete stress in the slab above the nominal 28 day strength, taken as 10 MPa
f_y	=	the yield strength of the beam flange
$M°_{b,i}$	=	the overstrength moment for beam i not considering any effect of axial force
	=	$\phi_{oms} M_s$;
ϕ_{oms}	=	overstrength factor from table 12.2.8(1) for MRFs and table 12.2.8(2) for D-braced EBFs with long links, as directed by 12.11.7.2
M_s	=	nominal beam section moment capacity to 5.2.
t_o	=	overall slab thickness
t_{ef}	=	the thickness of the concrete rib in direct contact with the column
	=	t_o when the rib is continuous along the beam (e.g. when the decking is parallel to the beam)
	=	t (from 13.1.2.5.1) otherwise
$\Sigma()$	=	indicates the summation of all beams having a moment connection at the joint in the direction of loading considered (either 1 or 2).

For calculating the seismic induced beam shears and the capacity design derived beam bending moment at the column centreline for each end of each beam, the column face overstrength moments at a joint with two beams framing into the column in the direction of loading considered shall be distributed between the beams on each side of the joint in proportion to their section moment capacities, M_s. For one beam, the column face overstrength moment shall be taken as $M°$.

(Amendment No.2, October 2007)

As an alternative to calculating $M°$, for I section beams framing into the column flange of I section columns, the overstrength actions from each beam including slab participation shall be given as $\phi_{omss}M_s$, where:

$\phi_{omss} = \phi_{oms} (1.0 + 1.08\ t_{ef}/d_b)$

for $t_{ef}/d_b \leq 0.4$

12.10.2.5
In the design of the beams to resist shear, the seismic shear component shall be determined from the overstrength moments.

12.10.2.6
When determining the design actions on the columns, it shall be assumed that the structure is displaced laterally so that the yielding regions form at the ends of all beams to give a yielding mechanism. The columns shall be designed for the overstrength actions from the beams from 12.10.2.3 or 12.10.2.4, as appropriate, and incorporating the dynamic magnification factor from 12.10.2.7, subject to these actions not exceeding the maximum actions required from 12.3.3.4, in conjunction with the actions from the permanent and combination imposed loads.

12.10.2.7
The dynamic magnification factor required is 1.2 for shear actions in columns, 1.0 for all other actions.

12.10.2.8
Design for concurrency in columns shall be in accordance with 12.8.4.

12.10.2.9
Design of connections between the beam and column shall be to 12.9.5.

12.10.2.10
Splices in columns shall be located clear of any potential yielding regions, in accordance with 12.9.6.1."

(Amendment No.2, October 2007)

12.11.1.1 (page 249)

Delete the second paragraph of this subclause and **substitute**:

"Category 1, 2 and 3 EBFs shall be designed and detailed in accordance with a rational capacity design procedure in compliance with 12.11.7 and the provisions of 12.11."

Delete the third paragraph of this subclause and **substitute**:
"Category 4 EBFs shall be designed and detailed in accordance with a rational design procedure and the relevant provisions of 12.11."

(Amendment No.2, October 2007)

12.11.3 Design requirements for category 1 EBF frames and components (page 250)

Delete the words "category 1" from the title.

(Amendment No.2, October 2007)

12.11.3.2 (page 250)

Delete the whole subclause and **substitute**:
"The relationship between the structure category and the category of the members comprising an EBF shall be as given in table 12.2.6, and applied as follows:
(1) The active link is the primary member
(2) The collector beam is also a primary member for compliance with 12.4, 12.5 and 12.6
(3) The column and the brace are secondary members."

(Amendment No.2, October 2007)

12.11.3.3.1 and **12.11.3.3.2** (page 250)
Delete "±0.09" and **substitute** "±0.08".

12.11.3.3.1 and **12.11.3.3.3** (page 250)
Delete "±0.045" and **substitute** "±0.03".

12.11.4.2 (page 252)
Delete "9 %" and **substitute** "8 %".

(Amendment No.2, October 2007)

12.11.4 Active link web stiffening requirements for category 1 EBFs (page 252)

Delete the title and **substitute**:

"**Active link web stiffening requirements for EBFs**"

(Amendment No.2, October 2007)

12.11.5 Connections between an active link and a column for category 1 EBFs (page 252)

Delete the title and **substitute**:

"**Connections between an active link and a column for category 1, 2 and 3 EBFs**"

(Amendment No.2, October 2007)

12.11.6 Lateral restraint for the active links of category 1 EBFs (page 253)

Delete the title and **substitute**:
"**Lateral restraint requirements for the active links of EBFs**"

Delete the first sentence of 12.11.6.2 and **substitute**:
"Top and bottom flanges of the active link between the braces for category 1, 2 and 3 EBFs shall be restrained at intervals as required for the yielding regions of the member from 12.6.2.3.1."

(Amendment No.2, October 2007)

12.11.7 (page 253)
Add the following new subclause and subclauses:

"**12.11.7 Capacity design requirements for EBFs**
The capacity design procedure for eccentrically braced frames shall meet the following requirements:

12.11.7.1
The active links are the primary members; the braces, collector beam and columns are the secondary members.

12.11.7.2
The braces and columns and all their connections shall be designed to resist the overstrength actions from the active links. The overstrength actions incorporate the overstrength factors, ϕ_{oms}, from table 12.2.8(2).

Also, for D-braced EBFs with $e \geq 3M_{sp}/V_w$ in 12.11.3.3.1, the overstrength moment generated at the face of the column shall incorporate the slab participation as described in 12.10.2.4.

The collector beams shall be designed to resist 0.8x the overstrength actions from the active links.

(Amendment No.2, October 2007)

12.11.7.3
When determining the design actions on the secondary members, it shall be assumed that the structure is displaced laterally so that yielding hinges form in all the active links to give a yielding mechanism. The secondary members shall be designed for actions from 12.11.7.2 and incorporating the dynamic magnification factor from 12.11.7.4, subject to these actions not exceeding the maximum actions required from 12.3.3.4, in conjunction with the actions from the permanent and combination imposed loads. "

12.11.7.4
The dynamic magnification factors required for design of columns are:
(a) For V-braced EBFs, 1.0 in all instances;
(b) For D-braced EBFs with columns that are category 2 members, 1.3 for determination of capacity design derived moments and axial forces and 1.2 for determination of capacity design derived shear forces;
(c) For D-braced EBFs with columns that are category 3 members, 1.5 for determination of capacity design derived moments and axial forces and 1.3 for determination of capacity design derived shear forces.

12.11.7.5
For the columns of category 1 and 2 EBF systems, the maximum capacity design derived design compression action N^*_{oc} on the column shall be:

$N^*_{oc} \le 0.8\ \phi N_s$ where $N^*_g / \phi N_s \le 0.3$

or $N^*_{oc} \le 0.7\ \phi N_s$ where $N^*_g / \phi N_s > 0.3$

12.11.7.6
Design for concurrency in columns shall be in accordance with 12.8.4.

12.11.7.7
When braces are welded to the active link/collector beam junction, this connection shall be analysed as rigid and the braces designed for the capacity design actions.

12.11.7.8
Splices in columns shall be located clear of any potential yielding regions, in accordance with 12.9.6.1.

12.11.7.9
Columns shall be designed as continuous past the incoming collector beam/active link at each storey of the EBF."

(Amendment No.2, October 2007)

12.12.3.1 Ultimate limit state design seimic loads (page 254)

Delete Step 1 and Step 2 and **substitute**:

"Step 1: Determine the horizontal design action coefficient, appropriate for the chosen structural ductility factor μ.

Step 2: Multiply C_d (T_1) by the appropriate value of C_s from tables 12.12.3 (1) – 12.12.3 (4)."

(Amendment No.2, October 2007)

12.12.5.2 (h) (page 260)

In line 2 **delete** the reference to "NZS 4203 subclause 4.7.3.1" and **substitute** "NZS 1170.5 subclauses 7.2.1 and 7.3.1".

(Amendment No.2, October 2007)

12.12.5.4 (g) (page 261)

Delete the existing sentence and **substitute**:

" (g) Concurrent action in columns forming part of a two-way seismic-resisting system shall be considered in accordance with 12.8.4. "

(Amendment No.2, October 2007)

12.12.7.2 Seismic design considerations (page 263)

Delete item (h) and **substitute** the following:

"(h) The nominal inelastic buckling load (N_{on}) of the notched region shall be calculated as $N_{on}=\pi^2 E_n I_n/(k_e L_n)^2$, where E_n shall be taken as 5000 MPa, I_n is the second moment of area of the notched section, k_e = 1.2 and L_n is the clear length of the notched section. N_{on} shall be taken as the minimum of N_{onx} or N_{ony} determined separately about each principal member axis.

N_{on} shall exceed 1.5 times the nominal member capacity of the brace as a whole, (N_c), determined in accordance with 6.3 and neglecting the effect of the notched region."

(Amendment No.2, October 2007)

13.2.1 Design method (page 270)

Retain the subclause number and title.

Delete the whole subclause and **replace** with:

"The design capacity of composite slabs on profiled steel sheet decking shall be to BS EN 1994-1-1:2004 Section 9 and the design of the deck itself for the wet concrete loading and construction stages shall be to BS EN 1993-1-3:2006. Loads and load arrangements on the slab shall be determined from the AS/NZS 1170 set. Other loads shall be determined from the documents specified from BS EN 1994-1-1:2004. Where these requirements specify different ultimate limit state load factors from those specified by AS/NZS 1170.0 and/or different strength reduction factors from those specified in table 13.1.2(1), the factor of safety (reliability) against failure shall be consistent with that specified by NZS 3404: Part 2 clause C3.1(c) (see NOTE (5)).

NOTE –

(1) For design of composite slabs on profiled steel sheet decking, BS 5950-4:1994 may be used instead of BS EN 1994-1-1:2004 and BS 5950-6:1995 (with 1999 corrigendum) may be used instead of BS EN 1993-1-3:2006.

(2) Design of the profiled steel sheet decking itself for the wet concrete and construction stages may be to AS/NZS 4600: 2005.

(3) The yield stress and tensile strength used in design for steels conforming to AS 1397 Grades 450, 500 and 550 shall be limited as specified in the standard used for design of the steel decking, but shall not be greater than 90 % of the minimum specified values from AS 1397.

(4) Design of composite slabs for concentrated loads closer than 500 mm to an end support requires rational design.

(5) This requirement will be met when the load factors are those specified by the standard used for the design of the composite slab or the design of the deck itself and the construction live loads are those specified by the standard used for the design of the composite slab. "

(Amendment No.2, October 2007)

13.2.2.1 (page 270)

Replace the first sentence of 13.2.2.1 with:

"Slabs shall be adequately reinforced to support all specified loads and to control cracking so as to meet the New Zealand Building Code performance requirements for durability, strength and fire safety."

(Amendment No.2, October 2007)

13.2.2.4 (page 271)

Delete the current subclause.

(Amendment No.2, October 2007)

13.2.4 Bases for design and construction (page 271)

Add the following new subclause 13.2.4.

"**13.2.4 Bases for design and construction**"

13.2.4.1
In the contract documents for construction the following bases for design shall be stated:
(a) The extent of propping of the deck and any supporting beams.
(b) The precambering of any supporting beams.
(c) The proposed method of screeding of the concrete surface, i.e. screeding to level or screeding to thickness.

13.2.4.2
Prior to construction commencing, if changes to any of these design bases are proposed, the adequacy of the floor slab and supports shall be determined to the approval of the Design Engineer or the Construction Reviewer.

13.2.4.3
When the deck and any supporting beams are unpropped, the maximum expected deflection, Δ_m of the floor system under the wet concrete shall be calculated and:
(a) The ponding effect shall be calculated when this deflection is greater than 1/10 of the specified effective thickness of concrete, h_e;
(b) The ponding effect shall be allowed for in calculating the unsupported deck span lengths and any propping requirements."

(Amendment No.2, October 2007)

13.3.2.1 Nominal shear capacity of connectors (page 272)

Delete Equation 13.3.2.1 and **substitute**:

"$q_r = \alpha_{dc} 0.13 \sqrt{f'_c} A_{sc} f_u \leq 0.8 f_u A_{sc}$ .... (Eq 13.3.2.1)

where f'_c is in MPa and there are at least 5 shear studs in the shear span.
All other variables are as defined in the Standard. "

(Amendment No.2, October 2007)

13.3.2.2.1 *General* (page 272)

Add a new subclause (f).

"(f) When the deck rib is oriented transverse to the steel beam and is continuous across the steel beam, the stud shall be placed in the centre of the rib, unless there is an upstand in the deck at the centre of the rib that prevents this. Where there is such an upstand, the studs shall be placed as close as practicable to the upstand and alternatively on the left hand side and on the right hand side of the upstand throughout the length of the span."

(Amendment No.2, October 2007)

13.3.2.2.2 *Reduction factors to be applied to stud capacity* (page 273)

Delete item (a) and **substitute:**

"(a) Steel deck ribs oriented transverse to the steel beam

The nominal shear capacity, q_r, shall be multiplied by the reduction factor α_{dc}

$$\alpha_{dc} = \left(\frac{0.70}{\sqrt{n_{rc}}}\right)\left(\frac{b_r}{h_{rc}}\right)\left(\frac{h_{sc}}{h_{rc}} - 1.0\right) \quad \text{(Eq 13.3.2.3)}$$

but for $n_{rc} = 1$, $\alpha_{dc} \leq 1.0$; for $n_{rc} = 2$, $\alpha_{dc} \leq 0.8$ and for $n_{rc} = 3$, $\alpha_{dc} \leq 0.6$

where
h_{rc} = nominal height of steel deck rib, mm
h_{sc} = length of stud connector after welding, mm, not to exceed the value (h_{rc} + 75) in calculations, although the actual length may be greater
b_r = width at the base of the concrete rib, mm
n_{rc} = number of stud shear connectors on a beam in one rib, not to exceed 3 in calculations for α_{dc} or for the number of studs installed in one rib."

Amendment No.2, October 2007)

13.3.2.3 Detailing requirements for shear connectors (page 273)

In line 2 of subclause (d), **change** "50 mm for an edge beam (L-beam)" **to** "the greater of 50 mm or $2.2d_{sc}$ for an edge beam (L-beam)" and in line 3 **change** "30 mm for an interior beam (T-beam)" **to** "the greater of 30 mm or $2.2d_{sc}$ for an interior beam (T-beam)".

(Amendment No.2, October 2007)

13.4.10 Longitudinal shear (page 283)

Delete the title of the subclause and **substitute:**

"**13.4.10 Longitudinal shear and post-cracking stud strength**

13.4.10.3 (page 283)
In 13.4.10.3, **delete** the words "longitudinal slab splitting" and **substitute** "longitudinal shear failure".

Add the following paragraph **after** the existing sentence:

Transverse reinforcement required to suppress longitudinal shear failure shall be placed across the relevant failure plane, anchored so as to develop the yield strength of the reinforcement and shall be uniformly spaced along the composite beam span."

(Amendment No.2, October 2007)

Add a new subclause 13.4.10.4 (page 283) as follows:

"**13.4.10.4**
Where the side cover to the stud, c_{do} < 10d_{sc} then, to ensure shear stud strength is maintained in the event of cracking along the line of the shear studs, transverse reinforcement, A_{rt} shall be provided as follows:

$$A_{rt} \geq 430 \frac{d_{sc}^2}{s_{sc}} \frac{M^*}{\phi M_{rc}} \text{ (mm}^2\text{/metre length)}$$

where
d_{sc} = diameter of shear stud (mm)
s_{sc} = average stud spacing along the line of the studs (mm)
c_{do} = the side cover to the stud, measured at the base of the stud
M^* = the maximum design moment along the line of shear studs (kNm)
ϕM_{rc} = the design moment capacity of the composite section (kNm)

This transverse reinforcement shall comply with the following:
(a) When placed as bars the bar diameter shall not exceed 16 mm.
(b) In all beams except internal primary beams, this transverse reinforcement shall be placed within a vertical height from the base of the stud not greater than 2.5d_{sc}.
(c) For internal primary beams, transverse reinforcement placed within the top 50 mm of the slab, including that for transverse flexural reinforcement, shall be considered contributing to A_{rt}.
(d) For edge beams, vertical hooked bars, where both the curve of the hook and the free end of the hook each extend more than 4d_{sc} mm past the centreline of the studs, shall be included. (Note that the bars may be inclined as required to fit within the depth of concrete available.)
(e) Decking that is transverse to the beam and continuous over the beam for at least 500 mm each side shall be considered contributing to A_{rt}.
(f) The development length for all bar or mesh reinforcement shall be as determined from NZS 3101 calculated for a reinforcement stress of 0.6f_{yr}. "

Appropriately placed reinforcement may contribute to both longitudinal shear and post-cracking resistance.

(Amendment No.2, October 2007)

13.5.4 (page 285)

In line 2 **change** "NZS 3101 section 8" **to** "NZS 3101:2006 clauses 9.3 and 9.4."

(Amendment No.2, October 2007)

13.8.2.3(a) (page 286)

In line 6 **change** "in accordance with 8.4.7.1(a) or 8.4.7.2(a) of NZS 3101:1995 as appropriate" **to** "in accordance with 10.3.10.5 or 10.3.10.6 of NZS 3101:2006 as appropriate,"

(Amendment No.2, October 2007)

13.8.2.3(b) (page 287)

In line 2 **change** "in accordance with NZS 3101:1995 subclause 8.5.4" **to** "in accordance with NZS 3101:2006 subclauses 10.4.6 and 10.4.7,"

(Amendment No.2, October 2007)

13.8.2.3(c) (page 287)

Retain figure 13.8.2.3 including subclauses (4) and (5) in (c) but **substitute** the existing subclause (c) with the following:

"(c) When applying NZS 3101:2006 subclauses 10.4.7.5.1 or 10.4.7.5.2 to composite columns, longitudinal reinforcement within the zone of influence shown by figure 13.8.2.3 need not be cross-linked."

(Amendment No.2, October 2007)

13.8.2.4.1 (page 288)

Make the following changes:
In item (a), **delete** "17.5 MPa" and **substitute** "20 MPa;"
In item (c), **delete** "shall conform with 8.4.7.1(a) of NZS 3101:1995" and **substitute** "shall conform with 10.3.10.5 of NZS 3101:2006".

(Amendment No.2, October 2007)

13.8.2.4.2 (page 288)

Make the following change:
In item (a) **delete** "17.5 MPa" and **substitute** "20 MPa; "

(Amendment No.2, October 2007)

13.8.3.3.2 (page 290)

In line three **delete** "from NZS 4203".

(Amendment No.2, October 2007)

14.3.6.1.2 (page 295)

Delete the subclause and **substitute**:

"The length of a bolt shall comply with (a) and either (b) or (c) as required:

(a) For all bolts, at least one clear thread shall show above the nut after tightening.
(b) For snug tightened bolts to 15.2.5.2(a), at least one clear thread run out shall be clear beneath the nut after tightening.
(c) For tensioned bolts to either 15.2.5.2 or 15.2.5.3, the minimum number of clear threads run out beneath the nut after tightening shall be:
   (i) Five threads for a bolt length (see table 15.2.5.2) up to and including 4 diameters;
   (ii) Seven threads for a bolt length over 4 diameters but not exceeding 8 diameters;
   (iii) Ten threads for a bolt length over 8 diameters. "

(Amendment No.2, October 2007)

14.4.4.2 Full contact splice (page 300)

Delete the subclause and **substitute:**

"If the ends of 2 butting lengths of a member, or the end of a member and the contact face of an adjoining cap plate or baseplate, are required to be in full contact, the maximum clearance between the contact surface and a straight edge shall not exceed 1 mm, and shall also not exceed 0.5 mm over at least 67 % of the contact area. The alignment of the butting surfaces shall be such that, when erected, the tolerances of 15.3.4 (d) are met. "

(Amendment No.2, October 2007)

15.1.2 Safety during erection (page 303)

Delete the existing NOTE and **substitute**:

"NOTE – Erection practices and procedures should comply with AS 3828."

(Amendment No.2, October 2007)

15.2.4.4 Retensioning (page 305)

Delete the existing subclauses 15.2.4.4.1 to 15.2.4.4.4 inclusive and **substitute**:

"Retensioning of bolts that have been fully tensioned shall not be permitted."

(Amendment No.2, October 2007)

15.2.5.2(a) (page 306)

After the first sentence **add** the following:

"This will require retightening to snug tight any bolts that become loose during the snug tightening of adjacent bolts."

(Amendment No.2, October 2007)

15.3.4 Column slice (page 310)
Add item (d) to the existing subclause:

"(d) For full contact column splices, the maximum clearance between the abutting surfaces shall not exceed:
- (i) (d/1000 + 1) mm;
- (ii) For gaps between (d/1000 + 1) and 6 mm, and the member is not part of a seismic frame, shims shall be required;
- (iii) For gaps between (d/1000 + 1) and 3 mm, and the member is part of a seismic frame, shims shall be required;
- (iv) For gaps exceeding (ii) or (iii) an engineering assessment shall be required."

(Amendment No.2, October 2007)

15.5.2 Grouting (page 312)

In the second paragraph **delete** "NZS 3108" and **substitute** "NZS 3104".

(Amendment No.2, October 2007)

17.4.3(b) (page 316)

In line 2 **delete** reference to "NZS 4203:Volume 2:Appendix C4.A or an equivalent test procedure" and **substitute** "ANSI/AISC 341-05 Appendices S or T or an equivalent test procedure".

(Amendment No.2, October 2007)

17.5.3(b) (page 317)

In line 1 **delete** reference to "NZS 4203:Volume 2:Appendix C4.A or an equivalent test procedure" and **substitute** "ANSI/AISC 341-05 Appendices S or T or an equivalent test procedure".

(Amendment No.2, October 2007)

Appendices, CONTENTS (page 319)

In Appendix H, at the end of title, **delete** the word "(informative)" and **substitute** "(normative)".
In Appendix J, at the end of title, **delete** the word ("informative") and **substitute** "(normative)".
Delete Appendix M.

(Amendment No.2, October 2007)

Appendix A – REFERENCED DOCUMENTS

NEW ZEALAND STANDARDS (page 320)

Add "NZS 1170.5:2007 Structural design actions, Part 5: Earthquake actions – New Zealand."

Add "NZS/AS 1657 Code for fixed platforms, walkways, stairways and ladders. Design, construction and installation."

Delete "NZS/BS 2573 Rules for the design of cranes, Parts 1 and 2."

Add "NZS 3104 Specification for concrete production."

Delete "NZS 3108 Specification for concrete production – Ordinary grade."

Add "NZS 3109 Concrete construction."

Delete "NZS 3402 Steel bars for the reinforcement of concrete."

Delete "NZS 3421 Hard drawn mild steel wire for concrete reinforcement."

Delete "NZS 4203 General structural design and design loadings for buildings."

Remove "P" from "NZS 4332P".

AUSTRALIAN AND JOINT NEW ZEALAND/AUSTRALIAN STANDARDS (page 320)

Add "AS/NZS 1170 set Structural design actions."

Delete "AS 1418 Cranes (including hoists and winches) and **substitute** AS 1418 set, Cranes, hoists and winches."

Add "AS/NZS 2312 Guide to the protection of structural steel against atmospheric corrosion by the use of protective coatings"

Add "AS 3828 Guidelines for the erection of building steelwork"

Add "AS/NZS 4671 Steel reinforcing materials"

Delete "AS/NZS 1110 ISO metric precision hexagon bolts and screws and **substitute** AS 1110 ISO metric hexagon bolts and screws – Product grades A and B."

Delete "AS/NZS 1111 ISO metric hexagon commercial bolts and screws and **substitute** AS 1111 ISO metric hexagon bolts and screws – Product grade C."

Delete "AS/NZS 1112 ISO metric hexagon nuts, including thin nuts, slotted nuts and castle nuts and **substitute** AS 1112 ISO metric hexagon nuts."

Delete "AS 1657 SAA Code for fixed platforms, walkways, stairways and ladders – Design, construction and installation."

Delete "SAA/SNZ HB 62 Code of practice for safe erection of building steelwork, " and **substitute** "AS 3828 Guidelines for the erection of building steelwork."

(Amendment No.2, October 2007)

BRITISH STANDARDS (page 322)

Delete "BS 6363 Specification for welded cold formed steel structural hollow sections," and **substitute** "BS EN 10219 Cold formed welded structural hollow sections of non-alloy and fine grain steels. Part 2: Tolerances, dimensions and sectional properties."

Add "BS 7668 Weldable structural steels. Hot finished structural hollow sections in weather resistant steels. Specification"

Add "BS EN 1993-1-3 Eurocode 3. Design of steel structures. General rules. Supplementary rules for cold-formed members and sheeting"

Add "BS EN 1994-1-1 Eurocode 4. Design of composite steel and concrete structures. General rules and rules for buildings"

Add "BS EN 10025 - - - Hot rolled products of structural steels.
- Part 1: General delivery conditions
- Part 2: Technical delivery conditions for non-alloy structural steels.
- Part 3: Technical delivery conditions for long products
- Part 4: Technical delivery conditions for the thermomechanical rolled weldable fine grain steels
- Part 5 Technical delivery conditions for structural steels with improved atmospheric corrosion resistance
- Part 6 Technical delivery conditions for plates and wide flats of high yield strength structural steels in the quenched and tempered condition"

Add "BS EN 10029 Specification for tolerances on dimensions, shape and mass for hot rolled steel plates 3 mm thick or above."

Add "BS EN 10210 Hot finished structural hollow sections of non-alloy and fine grain structural steels Part 1 Technical delivery requirements"

INTERNATIONAL STANDARDS (page 322)

Amend ISO 2566 to read "Steel – Conversion of elongation values"

Delete "ISO SST-1093-88 The application of an engineering critical assessment in design, fabrication and inspection to assess the fitness for purpose of welded structures."

Add "JIS G 3136 Rolled steel for building structure"

OTHER PUBLICATIONS (page 323)

Add "ANSI/AISC 341-05 Seismic Provisions for Structural Steel Buildings."

Add "HERA Report R4-133 New Zealand Steelwork Corrosion Coatings Guide."

Amend Fire protection for structural steel in buildings to be fourth edition.

Delete "Transit New Zealand Bridge Manual: Design and Evaluation " and **substitute** "Bridge Manual (SP/M/022) second edition, Transit New Zealand, 2003, incorporating amendments dated June and September 2004 and provisional amendment December 2004".

(Amendment No.2, October 2007)

Appendix B (page 326)

Add new note (7)

"(7) For structural categories 1 and 2 the serviceability limit state requirements of NZS 1170.5 need to be considered."

(Amendment No.2, October 2007)

Appendix C (page 327)

Delete clause C2.1 and **substitute:**

C2.1

"Requirements for painting of steelwork in most environments are given in AS/NZS 2312 (including Amendment No. 1) and HERA Report R4-133".

Extend the heading of C4 to read:

"**INACCESSIBLE SURFACES AND TREATMENT OF CUT EDGES**"

Add the following sentence to the end of C4 as a new paragraph:
"Cut edges shall be dressed before application of coatings to AS/NZS 2312 subclause 3.3.5.2".

OTHER PUBLICATIONS (page 330)

Delete the references to the HERA publications and **substitute**:

"HERA HERA Report R4-133 New Zealand Steelwork Corrosion Coatings Guide"

(Amendment No.2, October 2007)

Appendix H (page 337)

Delete line 1 and **substitute:**

"(This Appendix forms a normative part of this Standard.)"

(Amendment No.2, October 2007)

Appendix J (page 343)

Delete line 1 and **substitute:**

"(This Appendix forms a normative part of this Standard.)"

In clause J2 **delete** the first equation and **substitute**:

$$\left(\frac{R_w^*}{\phi R_{sb}}\right)+\left(\frac{N_w^*}{\phi N_{wo}}\right)+\left(\frac{V_w^*}{\phi V_v}\right)^2+\left(\frac{M_w^*}{\phi M_w}\right)^2 \leq 1$$

(Amendment No.2, October 2007)

Appendix M (page 352)

Delete Appendix M.

(Amendment No.2, October 2007)

STANDARDS NEW ZEALAND
PRIVATE BAG 2439
WELLINGTON 6020

NZS 3404:Parts 1 & 2:1997

STEEL STRUCTURES STANDARD

AMENDMENT No. 2 to Part 2

October 2007

REVISED TEXT

C1.3 DEFINITIONS (page 10)

Add a note at the beginning of clause C1.3 to say:

NOTE – Changes are required for alignment with the AS/NZS 1170 set and, in the case of design action, to correct an ambiguity in the Standard.

At the end of the definition for PRYING FORCE **delete** (refer to Appendix M).

(Amendment No.2, October 2007)

C2.6.4.5 Selection of welding consumables (page 24)

Add a new commentary subclause after C2.6.4.5.

C2.6.4.5.2
In table 4.6.1(A) of AS/NZS 1554.1 and in table 4.6.1(A) of AS/NZS 1554.5, the grading numbers for SAW, GMAW and FCAW consumables are followed by up to three alphabetic qualifiers. The previous provisions were only applicable to electrodes from one welding process; linking to the steel type makes the provisions applicable to all welding processes given in that table.

(Amendment No.2, October 2007)

C3.1 DESIGN (page 27)

Delete subclause (a) include subclause (1) to (4) and **substitute**:

(a) *Nominal loads*
The nominal loads are established for the return periods given in table 3.3 of AS/NZS 1170.0.

(Amendment No.2, October 2007)

C3.2.4, C3.2.4.1.2 and **C3.2.4.1.3** (pages 29-30)

Delete these subclauses and **substitute:**

C3.2.4
See the commentary to AS/NZS 1170.0 section C6.

(Amendment No.2, October 2007)

C3.3 ULTIMATE LIMIT STATE (pages 31 & 32)

On page 31, paragraph 2. line 3 **delete** 1.6Q and **substitute** 1.5Q
On page 31, paragraph 2, line 5 **delete** 1.6M^*_Q and **substitute** 1.5M^*_Q
On page 32, paragraph 4, **delete** 1.6Q and **substitute** 1.5Q
On page 32, paragraph 5, line 3 **delete** 1.6Q and **substitute** 1.5Q

(Amendment No.2, October 2007)

C3.4 SERVICEABILITY LIMIT STATE (page 33)

Add the following Note between the **C3.4** heading and **C3.4.1 General**:

NOTE – Not only is NZS 4203 replaced by the AS/NZS 1170 set, but many of the referenced publications given in this section have been superseded. It is beyond the scope of this amendment to give an updated reference list; contact HERA for advice on any updates.

(Amendment No.2, October 2007)

C3.4.6 Corrosion protection (page 34)

In paragraph 2 lines 2 and 3 **delete** Protection Guide for Steelwork Building Interiors – section 12 of (3.15) and **substitute** HERA Report R4-133 (3.21).

In the last line **delete** (3.20) and **substitute** (3.20, 3.21).

(Amendment No.2, October 2007)

REFERENCES TO SECTION C3 (page 35)
To the references for C3, **add** the following new reference:

3.21 HERA. 2005. New Zealand Steelwork Corrosion Coatings Guide, HERA Report R4-133. Published by HERA, Manukau City, New Zealand.

(Amendment No.2, October 2007)

Figure C4.3.1 – Distribution of live (variable) loading for determination of first-order column design moments (page 41)

Delete Note (4) and **substitute** the following:

(4) Q^*, G^* are the ultimate limited state design loads from the AS/NZS 1170.0. For Q^*, G^* together, the design load combination is $1.2G + 1.5Q$. For G^* alone, the design load is $1.2G$ in this instance, not $1.35G$.

(Amendment No.2, October 2007)

C4.7 LIMITS ON PLASTIC HINGE ROTATION IN YIELDING REGIONS OF MEMBERS (page 52)

Add a new commentary subclause after C4.7.

C4.7.2 Plastic hinge rotaion limits

Minor changes have been made in Amendment No. 2 following a review of the experimental data and to take into account the reserve of rotation capacity required to meet the 2500 year criterion set by NZS 1170.5. The design rotation capacity is always less than the minimum experimental result for each band. The ratio of mean/design rotation capacity is higher for category 3 members than for category 1 and 2 members, and generally increases as the axial load increases. There is very limited data on which these tables have been based.

Two of the experimental tests incorporated axial loading with a constant and a variable component, with the variable component being a function of the enforced lateral displacement. The two axial load regimes involved were $n = 0 \pm 0.3$ and $n = 0.3 \pm 0.3$, where n = fraction of nominal section capacity and $n > 0$ signifies compression. The same section size was also tested to the same displacement controlled lateral loading regime but with the axial load held at the constant value. The behaviour and inelastic rotation capacity of the speciman with the constant + variable axial loading in each instance was very similar to that of the specimen with constant axial loading of $n = 0$ and $n = 0.3$, respectively. This is consistent with the first principles model of axial shortening developed by MacRae (12.6). This shows that it is the average level of compression load over a full cycle of inelastic demand that is important in determining the available inelastic rotation capacity. Note (4) expresses this, however it has been amended to give the correct requirement (the average load over a cycle of response) when the earthquake component is non-symmetrical over the cycle. The previous note stated the seismic load component could be ignored, which is only applicable when the earthquake loading component is symmetrical.

(Amendment No.2, October 2007)

C4.9.1 General (page 54)

Add the following paragraph to the end of the subclause:

The elastic buckling load factor is calculated for second-order effect determination, typically using the moment amplification method of clause 4.4. As described in C4.4.2, this method becomes unreliable for $\lambda_c < 3.5$, however this limit is not expressly stated in the code clause 4.9 for calculating λ_c and may be overlooked, hence its inclusion.

(Amendment No.2, October 2007)

C8.4.3.2 Member slenderness (page 115)

Add the following paragraph to the end of the subclause:

A new and more accurate equation for end yielding criterion has been developed. Details are in the following report:
Plastic Hinge Location in Columns of Steel Frames, by Peng B. et al, University of Canterbury Civil Engineering Research Report, 2006.

(Amendment No.2, October 2007)

C9.1.3 Design of connections (page 127)

Add the following subclause after C9.1.3:

C9.1.3.5
Unstiffened plates designed to transfer compression forces fail in a sway mode. If there is any eccentricity in the path of the compression force transfer, as will occur due to construction out of tolerance or when there is eccentricity between the plates transferring the compression, the connection will commence sway as soon as compression load is applied. The peak compression load that can be reached will be sensitive to construction tolerances and second-order effects.

For eccentrically loaded cleats in compression, involving one plate from the supporting member bolted into one plate from the supported member, a design procedure is presented in *Eccentric Cleats in Compression and Columns in Moment-resisting Connections*, HERA Report R4-142, Manukau City, New Zealand, 2007.

(Amendment No.2, October 2007)

C9.1.4 Minimum design actions on connections (page 128)

Add the following subclause after C9.1.4:

C9.1.4.1 Minimum design actions on connections not subject to earthquake loads or effects

For structures which incorporate a rigid floor diaphragm, that is composite with the supporting beams and in direct contact with the columns, the requirement of new subclause 9.1.4.1(c) can be deemed to be satisfied.

(Amendment No.2, October 2007)

C9.3.2.1 Bolt in shear (page 131)

In line 3 **delete** AS/NZS 1111 and **substitute** AS 1111.

(Amendment No.2, October 2007)

C10.2.1 (page 160)

In paragraph 2, line 2 and line 4 **delete** NZS 4203 and **substitute** AS/NZS 1170.2 and in line 5 **delete** 350 and **substitute** 500.

(Amendment No.2, October 2007)

C12.2.2.1 Structural performance factor (page 173)

Delete the whole subclause and **substitute**:

The changes to S_p are to align with NZS 1170.5 and be consistent with treatment of systems of similar ductility from NZS 3101:2006. See C4.4 of that Standard for the background to the new requirements for S_p.

(Amendment No.2, October 2007)

C12.2.3 Classification of structural systems (page 174)

Add the following commentary subclause after C12.2.3.

C12.2.3.1 Categories of ductility demand
These changes are to make the performance requirements more quantifiable, for compliance with NZS 1170.5.

(Amendment No.2, October 2007)

C12.2.3.3 Application of structural classifications (page 174)

Add the following two paragraphs :
The changes are made to ensure consistent performance, to meet the requirements placed on the materials standards by NZS 1170.5 and to ensure that the rotation demand on members of the seismic-resisting system will not exceed the capacities of tables 4.7 in the cases where an explicit check for rotation using this table is not required.

The restriction on the system category that can be used for two seismic-resisting systems in a concurrent or dual configuration introduced in 12.2.3.3 brings into the Standard an existing provision in HERA Report R4-76 (12.3) and in the existing commentary clause for Dual Systems, being the third paragraph of Commentary Clause C12.13.

(Amendment No.2, October 2007)

C12.2.4 Structural displacement ductility demands (page 175)

Add the following paragraph:

The changes are made to ensure consistent performance, to meet the requirements placed on the materials standards by NZS 1170.5 and to ensure that the rotation demand on members of the seismic-resisting system will not exceed the capacities of tables 4.7 in the cases where an explicit check for rotation using this table is not required.

(Amendment No.2, October 2007)

C12.2.6 Relationship between structure category and member category (page 175)

Add the following paragraphs:

The changes are made to ensure consistent performance, to meet the requirements placed on the materials standards by NZS 1170.5 and to ensure that the rotation demand on members of the seismic-resisting system will not exceed the capacities of tables 4.7 in the cases where an explicit check for rotation using this table is not required.

These provisions have been developed on the assumption of rigid plastic deformation and rigid foundations, both of which generate maximum plastic rotation demand in the elements of the superstructure subject to inelastic demand.

Short steel members may yield in shear, rather than, or in addition to, flexure. Studies on steel shear links indicate that principally shear yielding occurs in members with a length less than $3M_p/V_w$. These members have a lower strength than those subject to flexural yielding only. Members detailed as active links according to clause 12.11 have the design strength given in subclause 12.11.3.5 and they can undergo rotations greater than that for regular members in flexure according to subclause 12.11.3.3.1.

For case number 4, the required column category has been determined through calculating the plastic rotation associated with inelastic response being concentrated into the lowest storey, for a building at the critical height, when the inelastic drift limit from NZS 1170.5 is reached at the top of the frame and selecting the required category from table 4.7(3) for a design axial compression force of $0.5\phi N_s$. In this case Category 2 is required. Use of table 4.7 gives sufficient reserve of strength to meet the 2500 year criterion set by NZS 1170.5.

For case number 5, there is no inelastic demand on the columns at the design ultimate limit state and the 2500 year criterion is accommodated principally through the use of S_p = 0.9 for these non brittle elastically responding systems. Additional protection for multi-storey building columns is afforded by the requirements for column design, which, for axial loads > $0.15\phi N_s$, will dictate that the column cross section elements are typically below the yield slenderness limit for section efficiency and hence will provide some dependable inelastic rotation capacity.

(Amendment No.2, October 2007)

C12.3.2.3.2 (page 178)

Delete the last paragraph plus heading and **substitute**:

A simple method of calculating μ_{act} is through Equation C12.3.2(4):

$$\mu_{\text{act}} = \mu\left(\frac{\sum S_{\text{E}}^{*}}{\sum \varphi R_{\text{u,E}}}\right) \geq 1.0 \quad \text{(Eq. C12.3.2(4))}$$

where

μ = the design ductility

S_{E}^{*} = design actions from analysis for the design ductility, μ, at the yielding regions of the primary members. These design actions which include redistribution were applied.

$\phi R_{\text{u,E}}$ = design capacity of the primary members at the yielding regions

$\sum$ = summation of all yielding regions in the yielding mechanism.

Use of μ_{act} in NZS 1170.5

Once μ_{act} is determined, it is important that the increased seismic design loads resulting from the reduction in μ, from μ_{des} to μ_{act}, are applied in determining the elastic deflections to be increased in accordance with NZS 1170.5 subclause 7.2.1 to give the inelastic deflections. The decreased ductility demand will also be beneficial in reducing any *P*-delta effects from clause 6.5 of NZS 1170.5 and in meeting the concurrency requirements of 12.8.4.

(Amendment No.2, October 2007)

C12.3.3.4 Maximum design actions for members of seismic-resisting systems (page 180)

Add the new commentary subclause C12.3.3.4 before subclause C12.3.4 as follows:

When the upper limit design actions on secondary members are less than the capacity design derived design actions based on primary member overstrength, the level of protection against inelastic demand afforded to the secondary members is reduced. When setting the value of μ_{act} at which μ_{max} changes from nominally ductile to elastic, then if the ratio $\frac{\mu_{\text{act}}}{\mu_{\text{max}}} \geq \phi_{\text{oms}}$, the same strength hierarchy against inelastic response in the secondary members for the overall system will be afforded by using the upper limit actions as is given by the capacity design process. If the ratio $\frac{\mu_{\text{act}}}{\mu_{\text{max}}} < \phi_{\text{oms}}$, then using the upper limit actions for the secondary members will afford less strength hierarchy protection. This approach has been used to assess the adequacy of μ_{act} = 1.5 as the change-over for μ_{max} from nominally ductile to elastic, as given in the 1997 Standard, and has shown that the change-over value should be increased to $\mu_{\text{act}} \geq$ 1.8. The same applies in 12.9.1.2.2 (4) (a) and (b).

(Amendment No.2, October 2007)

C12.3.4 Design and detailing of members and connections of associated structural systems (page 180)

Add the following paragraph to the end of the subclause:

Changes have been made to the maximum design actions to incorporate the 2500 year criterion from NZS 1170.5 and also to enhance the level of protection to lower ductility capacity systems.

(Amendment No.2, October 2007

C12.4 MATERIAL REQUIREMENTS (page 181)

Delete the last two paragraphs and **substitute** with:

Significant changes have been made to the material requirements for seismic design, for the following reasons:

(a) The requirements for elongation, yield ratio and maximum yield stress are based on the actual steel to be used. This requires a check to be made on these properties in the supply chain for the steel to go into the seismic-resisting system.

(b) Research has shown that grade 350 steel is now suitable for category 1 applications, provided it meets all of the criteria of this table.

(c) The requirement for a minimum Charpy impact energy requirement has been shown to be necessary (12.66). Steels meeting these criteria will also have good elongation performance.

(Amendment No.2, October 2007)

C12.8.3.1 Limitations on axial force (page 184)

Add the following paragraphs to the end of the subclause:

The application of this subclause has been elaborated on to remove uncertainties as to which design actions are required to be used for each of (a) to (c).

A new and more accurate equation for end yielding criterion (Equation 12.8.3.1) has been developed. Details are in (12.63) and these replace the first paragraph of the existing commentary on the background to 12.8.3.1(b).

(Amendment No.2, October 2007)

C12.8.4 Concurrent action on columns (page 186)

Delete the existing commentary and **substitute**:

The new subclause makes the concurrency provisions consistent with the concurrency requirements of NZS 1170.5 and covers all options that are allowed under the new restrictions of 12.2.3.3 on the category of each system in a concurrent configuration.

(Amendment No.2, October 2007)

C12.9.1.2 Design actions for connectors and connection components (page 187)

Add the following paragraph to the existing commentary under the subheading *Further background to subclause (4)*.

The changes to maximum actions are to incorporate the 2500 year criterion from NZS 1170.5 and also to enhance the level of protection to lower ductility capacity systems. Also 12.9.1.2.2(4) is supposed to represent the upper limit earthquake induced actions, however the second half of the sentence contradicts this by stating that 12.9.2 governs if these actions are greater. The amendment to this subclause and to 12.9.2 corrects this error.

(Amendment No.2, October 2007)

C12.9.5.2 (b)(ii)(2) **Design actions from beams** (page 191)

Add the following immediately above the heading **Suitable method for determination of** V_G.

An element of slab participation is to be considered for panel zone design, when required by 12.10.2.3. See (12.65) for background to the provisions.

Amendment No.2, October 2007)

C12.9.7 Design of gusset plates (page 198)

Add a new commentary subclause after figure C12.9.7.

C12.9.7.4
A gusset plate which connects into both a beam and a column member will be stretched when the angle between these two members tries to increase under inelastic seismic action. This requirement is to ensure the connection between gusset plate and support does not fail in this situation.

For gusset plates that are welded into the supporting member (either the beam or the column), this can be achieved by sizing the weld between the gusset plate and the member to develop the design tension capacity of the gusset plate gross cross section at the connection.

For gusset plates that are bolted into the supporting member, this can be achieved by either designing the connection to accommodate opening of the joint angle under inelastic seismic action or by designing the bolted connection to develop the design tension capacity of the gusset plate gross cross section at the connection.

(Amendment No.2, October 2007)

C12.10.2 Design procedure (page 198)

Add the following paragraph:

NZS 1170.5 requires the materials standards to specify the principles and key concepts required for capacity design. These concepts are already incorporated into the step by step requirements of HERA Report R4-76, with the exception of the slab participation factor now required by 12.10.2.3 and specified in 12.10.2.4.

(Amendment No.2, October 2007)

C12.10.2 Design procedure (page 198)

Add a new commentary subclause after C12.10.2:

C12.10.2.3
The slab participation is required when determining the overstrength actions on the column member for combined actions design when the slab is not isolated from the column.

Suitable details for isolating the slab from the column are given in section 4 of (12.64).

(Amendment No.2, October 2007)

C12.10.2 Design procedure (page 198)

Add a new commentary subclause after C12.10.2.3:

C12.10.2.4

This method of accounting for slab participation in the overstrength joint moment is based on the assumption that the slab is infinitely rigid and strong axially, carrying force due to compression contact of the concrete against the column over a defined contact area. The slab contributes to both positive and negative moments in the adjacent beams, as in the negative moment case the tension force developed in the reinforcement has to be transferred into the column through the concrete slab. The distance between the centroid of compression contact area and that of the steel section generates the lever arm for determination of the moment induced by this slab action. The slab compression force has to be balanced by an axial tension in the steel beams and this reduces the beam moment capacity. The calculation of joint overstrength, M°, takes both factors into account.

The determination of bearing width against the column face is based on work by Civjan et al (12.59) for beams framing into the strong axis of an I section, with the same approach being applied to the other column types of cross section listed.

The value of b_{sef} for circular columns is from applying the principles of Blodgett (12.60) for distribution of transverse load from beam flanges into a circular hollow section column member. They are based on 80 % of the concrete compression force being resisted by the compression contact face of the slab onto the column.

Values of concrete stress at the concrete face ranging from less than 0.85 f'_c to 1.80 f'_c have been found in past tests Civjan et al. (12.59). Du Plessis and Daniels (12.61) and Lee and Lu (12.62) found that the concrete stress is usually 1.30 f'_c or less. Values less than 1.30 result for partially composite slabs so a value of 1.30 is recommended for New Zealand application until tests on locally constructed slabs on beams are available.

The variable f'_{cos} takes account of the increase in concrete strength above the minimum specified that occurs in practice, due to both short and long term effects for a slab which has different compaction and curing than large concrete members.

It should be noted that steel beams have an upper limit on the moment that they can resist as a result of beam axial tension and compression yielding which is different from reinforced/ prestressed concrete beams where large moments may result due to the slab.

For I section beams framing into the column flange of I section columns, the results from using the general equations have been determined for a typical range of beam and column sizes and ratios of slab effective thickness to beam depth. These have then been expressed as a function of the enhancement to the overstrength factor, ϕ_{oms}, against the ratio of slab effective thickness to beam depth. This function can be expressed as an enhancement to the overstrength factor ignoring slab participation in the format shown with only 3 % loss of accuracy within the practical range of t_{ef}/d_b. The enhanced overstrength factor, ϕ_{omss}, is used in place of ϕ_{oms} for determining the seismic induced beam shears and the column design moments, when applying the provisions of HERA Report R4-76 (12.3).

The slab participation is required when determining the overstrength actions on the column for flexural design. It may not be required for calculation of the column panel zone shear strength, as the slab effect on the panel zone is neutral in that it may increase both the panel zone shear forces and the panel zone shear strength.

(Amendment No.2, October 2007)

When applying the capacity design provisions of section 6 of HERA Report R4-76 (12.3), if M° is calculated directly, then it is used in place of M°_{beam} in step 5.1 and subsequent steps of section 6. Where two beams frame into the column, M° must be distributed into the beams as directed by the clause. If the enhanced overstrength factor option for I section beams framing into the flanges of I section columns is used, then this enhanced overstrength factor is used instead of ϕ_{oms}.

The effect of the slab is to increase overstrength demands on the columns. The aim of the slab participation factor is to incorporate this slab effect into the capacity design derived design actions on the columns, such that the inelastic demand on the columns is not underestimated due to unaccounted-for slab action. Advanced finite element modelling of a representative subassemblage with and without the slab has shown (12.65) that the proposed factor achieves this aim, both for the column above and below the beam (subclause 12.10.2.4) and for the panel zone (subclause 12.9.5.2 (b) (ii) (2)).

Example:

Beams framing into the column (one from each side):
460 UB74.6, f_y = 300 MPa, M_s = 498 kNm, $2M_s$ = 996 kNm,
$A_g f_y$ = 9520 mm^2 x 300 MPa = 2,856 kN, 2 $A_g f_y$ = 5,712 kN
MRF is category 2, Overstrength factor, bare steel ϕ_{oms} = 1.15

Column: 310 UC 118, f_y = 300 MPa

Frame configuration: Interstorey height is 3.6 m, and bay width is 7.0 m

Slab: A slab depth of 120 mm on a metal deck which runs in the same direction as the beam, thus forming a solid concrete rib down the beam. The concrete strength is f'_c = 30 MPa.

N_{slab} = $min\{1.3 t_{ef} b_{sef}(f'_c + 10); \Sigma A_s F_y\}$
= min{1.3*120 mm*307 mm*40 MPa = 1,916 kN; 5,712 kN}
= 1,916 kN

ΣM°_{i} = min{1.18 x (1 – 1916/5712) x 996 x 1.15; 996 x 1.15}
= min{898; 1145} = 898 kNm

M° = 898 + 1,916 (457/2+120-120/2) x 10^{-3} = 1451 kNm

Overstrength factor due to the slab effect alone is:
ϕ_{oss} = $M^{\circ} / \Sigma M^{\circ}_{b,i}$ = 1451/1145 = 1.27

Overstrength factor for the beam with slab participation
ϕ_{omss} = $M^{\circ}/\Sigma M_s$ = 1451/996 = 1.46

Alternatively,
ϕ_{omss} = $\phi_{oss}\Sigma M^{\circ}_{b,i}/\Sigma M_s = \phi_{oss}\Sigma(\phi_{oms}M_s)/\Sigma M_s = \phi_{oss}\phi_{oms}\Sigma M_s/\Sigma Ms = \phi_{oss}\phi_{oms}$
= 1.27x 1.15 = 1.46

(Amendment No.2, October 2007)

C12.10.2 Design Procedures (page 198)

Add a new commentary subclause after C12.10.2.4:

C12.10.2.6
Inelastic time history studies carried out by MacRae (12.6), Clifton (12.64) and others have shown that a soft-storey mechanism does not develop in a steel moment resisting frame complying with the design requirements of this Standard and HERA Report R4-76 (12.3). For this reason the dynamic magnification factor for moment is 1.0. However, MacRae proposed (12.6) a shear magnification of 1.2 to limit the possibility of undesirable shear yielding in steel columns with unstiffened webs.

These studies did not incorporate the strengthening effect of the slab when this is not isolated from the columns and this has been addressed through the slab participation factor in 12.10.2.3.

(Amendment No.2, October 2007)

C12.11 DESIGN OF ECCENTRICALLY BRACED FRAMED SEISMIC-RESISTING SYSTEMS (page 200)

Add the following paragraph immediately under the heading for C12.11:

NZS 1170.5 requires the materials standards to specify the principles and key concepts required for capacity design. These concepts are already incorporated into the step by step requirements of HERA Report R4-76 and are presented in the elaboration of subclauses 12.11.1.1, 12.11.3.2 and new subclauses in 12.11.7.

(Amendment No.2, October 2007)

C12.11.3.3 (page 202)

Add a new paragraph to the end of the subclause:

The reduction in allowable plastic rotation limits for active links arises from the need to address non collapse in the 2500 year event from NZS 1170.5 and is consistent with the limits in the current American and European seismic design requirements.

(Amendment No.2, October 2007)

C12.11.4 Active link web stiffening requirements for category 1 EBFS (page 203)

Add a new sentence to the end of the subclause:

The changes are to make these subclauses applicable to more than category 1 EBFs.

(Amendment No.2, October 2007)

C12.11.5 Connections between an active link and a column for category 1 EBFs (page 203)

Add a new sentence to the end of the subclause:

The changes are to make these subclauses applicable to more than category 1 EBFs.

(Amendment No.2, October 2007)

C12.11.6 Lateral restraint requirements for the active links of category 1 EBFs (page 204)

Add a new sentence to the end of the subclause:

The changes are to make these subclauses applicable to more than category 1 EBFs.

(Amendment No.2, October 2007)

Add a new commentary subclause after C12.11.6 (page 204)

C12.11.7 Capacity design requirements for EBFs

NZS 1170.5 requires the materials standards to specify the principles and key concepts required for capacity design. These concepts are already incorporated into the step by step requirements of HERA Report R4-76 (12.3), with the exception of the slab participation factor required for D-braced systems with long links by 12.11.7.2.

The collector beam may undergo inelastic action when the active link is fully inelastic. This is allowed for in (12.13) by designing the collector beam to resist 80 % of the capacity design derived design actions, rather than 100 % required for all other secondary members, as specified by 12.11.7.2. The collector beam is required by 12.11.3.2 (2) to have suitable properties to resist this inelastic demand.

The dynamic magnification factors in 12.11.7.4 are obtained by MacRae(12.6). More recent studies (12.63) of EBFs designed to this Standard and HERA Report R4-76 (12.3) have confirmed the adequacy of these factors.

(Amendment No.2, October 2007)

C12.12.3 Design seismic loads for CBFs with bracing effective in tension and compression (page 205)

In paragraph1, line 3, **delete** "clause 4.6 of NZS 4203" and **substitute** "clause 5.2 of NZS 1170.5".

(Amendment No.2, October 2007)

Add a new commentary subclause after C12.12.7 (page 210)

C12.12.7.2 Seismic design considerations
The change to k_e reflects the sway failure mode of the notched region.

(Amendment No.2, October 2007)

REFERENCES TO SECTION C12 (page 212)

Alterations to references are as follows:

Change reference 12.3 **to** be:

12.3 Feeney, M.J. and Clifton, G.C. Seismic Design Procedures for Steel Structures. HERA, Manukau City, 1995, HERA Report R4-76; to be read with Clifton, G.C.; Tips on Seismic Design of Steel Structures. Notes from Presentations to Structural Groups mid-2000; HERA, Manukau City, 2000.

Change reference 12.5 from NZS 4203:1992 **to** NZS 1170.5:2004 Structural design actions.

Add the following references:

12.59 Civjan, S. A. Engelhardt M. D. and Gross J. L. Slab Effects in SMRF Retrofit Connection Tests, J. Struct. Engrg, ASCE, pp.230, March 2001.

12.60 Blodgett, O.W. Weld Failures : They Could be the Result of Violating Simple Design Principles, Welding Journal, March & April 1982.

12.61 Du Plessis, D. P. and Daniels, J. H. Strength of Composite Beam-to-Column Connections, Report No. 374.3, Fritz Engineering Lab, Lehigh, Bethlehem, PA, 1972.

12.62 Lee, S. J. and Lu, L. W. Cyclic Tests of Full-scale Subassemblages. J. Struct. Engrg, ASCE, 115(8), pp.1977-1998, 1989.

12.63 Plastic Hinge Location in Columns of Steel Frames, by Peng, B. et al. University of Canterbury Civil Engineering Research Report, 2006.

12.64 Clifton, G.C. Semi-rigid Joints for Moment-resisting Steel Framed Seismic-resisting Systems. HERA Report R4-134, HERA, Manukau City, New Zealand, 2005.

12.65 MacRae, G,. Clifton, G.C. and Mago, N Overstrength Effects of Slabs on Demands on Steel Moment Frames Proceedings of the 2007 Pacific Structural Steel Conference, Wairakei, New Zealand; HERA, Manukau City, New Zealand, 2007.

12.66 Hyland, C. Steel Selection for Seismic Purpose, Session 2, Steel Structures Seminar, 2005 SCNZ/HERA, Manukau City, New Zealand, 2005.

(Amendment No.2, October 2007)

C13.1.2.1 (page 217)

In the second paragraph, **retain** the references to NZS 4203.

Delete the last paragraph and **replace** with the following:

"These strength reduction factors do not need changing with the adoption of the AS/NZS 1170 set."

(Amendment No.2, October 2007)

C13.2.1 Design methods (page 220)

Delete the existing subclause and **replace** with:

The two Eurocode standards provide the most up to date design requirements and have the widest scope of application. However, the most recent edition of the British standard for the design of the composite deck and the associated standard for design of the bare steel deck are still applicable and so these options are given as NOTE (1).

AS/NZS 4600 is a suitable standard for design of the deck itself for the wet concrete and construction stages and this option is given as NOTE (2). AS/NZS 4600 is also cited in B1/VM1. The steel used for decks is typically a cold-rolled steel to AS 1397 with limited ductility, hence the limits on design yield stress and tensile strength in NOTE (3).

Neither BS 5950-4 or BS EN 1994-1-1 cover the design of the slab for concentrated loads close to the support, which is the reason for NOTE (4). One option in this situation is to ignore the effect of the decking and design the slab for moment and shear to NZS 3101. See design guidance including software in (13.51). This situation principally applies at decking end supports which are also at the free edge of the slab.

(Amendment No.2, October 2007)

C13.2.2 Slab reinforcement (page 220)

Delete all but the last three paragraphs of C13.2.2 and **replace** these with:

The pattern and extent of cracking in a composite slab on profiled steel sheet deck is different to that in a reinforced concrete slab, hence the minimum reinforcement provisions for reinforced concrete slabs, such as given in NZS 3101) are not directly applicable.

Minimum reinforcement requirements are needed to control cracking due to shrinkage and temperature effects and these requirements are applicable to the depth of concrete above the ribs (i.e. the cover slab, t_o-h_{rc}). Reinforcement is also required to control cracking where the slab spans over supporting beams or into end supports.

The 1997 edition presented recommendations from BS 5950-4:1994 for interior applications only. However these have been superseded by latter standards and have been shown in practice to allow undesirable crack widths to develop. They also apply only to slabs that are fully enclosed within a building except during construction and this scope of application needs to be widened.

Working with the provisions of BS EN 1992-1-1:2004, AS 3600:2001, NZS 3101:2006 and BS 5950-4, the following minimum reinforcement requirements for composite slabs on steel deck are as follows (except where specific design for crack control is used as an alternative to the prescriptive provisions of (A.1, A.2) and (B)):

(A) Reinforcement parallel to the span of the decking

(A.1) Over the supporting beams of nominally continuous slabs

Where the slab is designed as simply supported, the area of reinforcement over the supporting beams shall be the greater of:

(a) 0.2 % of the cross-sectional area above the ribs (i.e. based on the depth (t_o-h_{rc})) where the slabs are unpropped and 0.4 % of the cross-sectional area above the ribs where the slabs are propped;
(b) That required from (B) below for the degree of crack control and exposure classification;
(c) That required for fire emergency conditions.

(A.2) Over the supporting beams of continuous slabs where specific control of crack widths is not required

(Amendment No.2, October 2007)

The area of reinforcement shall be the greater of:
(a) That required for flexural strength for ambient temperature or fire emergency conditions.
(b) 0.2 % of the cross-sectional area above the ribs where the slabs are unpropped and 0.4 % of the cross-sectional area above the ribs where the slabs are propped.
(c) That required from (B) below for the degree of crack control and exposure classification.

(A.3) Over the supporting beams of continuous slabs where specific control of crack widths is required

(a) Scope of application: this is required for critical tensile zones in exposure classification C from NZS 3101 or to critical tensile zones in exposure classifications A2, B1 or B2 where areas of reinforcement less than those specified for strong crack control in (B.3) are desired.
(b) A critical tensile zone is a region of slab where the design bending moment at the serviceability limit state, M_s^*, calculated using short term live loads, exceeds the uncracked moment capacity, M_{cr}. M_{cr} is calculated using gross cross section properties and a flexural tensile strength of concrete equal to $0.4\sqrt{f_c'}$.
(c) Using the cracked section properties, the stress, f_s, in the largest diameter of reinforcing bar subject to M_s^* shall be determined.
(d) The stress from (c) shall not exceed the lesser of (0.8 x the yield stress) for the reinforcement or that given in the table below:

Nominal bar dia. (mm)	**Maximum steel stress** (MPa)
≤ 6	375
6 - 8	345
10	320
12	300
16	265
20	240

(e) The minimum area of reinforcement in the critical tension zone (mm^2/m width of slab) shall be the greater of that required from (A.2) and:

$$A_{r\,min} = 1.8\frac{A_{ct}}{f_s}$$

where
A_{ct} = the area of concrete in the tension zone of the uncracked section (mm^2/m width of slab)
f_s = the stress from (d) above (MPa).

(f) The crack width shall be determined from NZS 3101 subclause 2.4.4.6 and assessed against suitable recommended crack widths, such as given in NZS 3101 table C2.1.

(A.4) Simply supported slabs and the positive moment regions of continuous slabs

The area of reinforcement placed above the ribs of the deck in these regions shall be the greater of:
(a) That required for fire emergency conditions and
(b) That required from (B) below for the degree of crack control and exposure classification.

(Amendment No.2, October 2007)

(B) Reinforcement transverse to the span of the decking

(B.1) For slabs fully enclosed within the building except during construction:

$A_{r,\,min}$ = $CR\ (t_o - h_{rc})$

where

$A_{r,\,min}$ = minimum area of reinforcement required (mm^2/m width of slab)

t_o = overall thickness of slab (mm)

h_{rc} = height of deck ribs (mm)

CR = 1.75 when a *minor* degree of control against cracking is required

CR = 3.5 when a *moderate* degree of control against cracking is required

CR = 6.0 when a *strong* degree of control against cracking is required.

(B.2) For slabs in exposure classification A2 from NZS 3101:

$A_{r,\,min}$ = $CR\ (t_o - h_{rc})$

where

$A_{r,\,min}$ = minimum area of reinforcement required (mm^2/m width of slab)

t_o = overall thickness of slab (mm)

h_{rc} = height of deck ribs (mm)

CR = 3.5 when a *moderate* degree of control against cracking is required

CR = 6.0 when a *strong* degree of control against cracking is required.

(B.3) For slabs in exposure classifications B1, B2 and C from NZS 3101:

$A_{r,\,min}$ = $CR\ (t_o - h_{rc})$

where

$A_{r,\,min}$ = minimum area of reinforcement required (mm^2/m width of slab)

t_o = overall thickness of slab (mm)

h_{rc} = height of deck ribs (mm)

CR = 3.5 when a *moderate* degree of control against cracking is required (but see (D.2) for additional requirements)

CR = 6.0 when a *strong* degree of control against cracking is required.

(B.4) For slabs in exposure classification U from NZS 3101, specific design is required.

(C) Reinforcement spacing, type and bar size and concrete quality

(C.1) Reinforcement for shrinkage and temperature control only may be Ductility Class L, N or E to AS/NZS 4671. Reinforcement for flexural strength where inelastic response is not required may be Ductility Class L, N or E. Reinforcement for fire resistance shall be Ductility Class N or E or as specified by the fire design procedure. Reinforcement for earthquake resistance in potential inelastic regions shall be Ductility Class E.

(C.2) The centre to centre spacing of bars in each direction for crack control shall not exceed the lesser of $3(t_o - h_{rc})$ or 300 mm. The diameter of the largest bar used for crack control should not be greater than 2 times the diameter of the smallest bar used for crack control.

(C.3) The cover required to the top reinforcement specified in (A) or (B) above shall comply with NZS 3101 tables 3.6 or 3.7.

(C.4) The clearance between the transverse reinforcement required from (B) above and the top of the deck ribs shall be not less than the maximum size of the aggregate used in the slab concrete, except at splices in this reinforcement.

(C.5) Laps to reinforcement and curtailment of reinforcement shall comply with NZS 3101.

(Amendment No.2, October 2007)

(C.6) Potential cracking due to plastic shrinkage of concrete shall be controlled by specification. Concrete placement and curing shall be to NZS 3109. See also sections 2.6 and 2.7 of HERA Report R4-107:2005 *Composite Floor Construction Handbook* for good practice requirements in these areas. Poor quality concrete production, placement or curing will jeopardise achieving the expected crack control for the area of reinforcement specified.

(D) Description of the crack control terms

What each of the terms minor, moderate and strong mean in terms of crack control as specified in (B) above is described below:

(D.1) *Strong* crack control will limit crack widths to generally < 0.3 mm. This is necessary for durability in certain circumstances as specified in (B) and is expected to be acceptable for visual appearance in exposed slabs.

(D.2) *Moderate* crack control will limit crack widths to generally < 0.5 mm. For cracks of this width, crack sealing admixtures are expected to dependably seal any cracks that develop. Without such admixtures, this crack width will not be suitable for durability in external environments B1, B2 or C to NZS 3101. This width of crack is not expected to be acceptable for high quality visual appearance.

(D.3) *Minor* crack control will give crack widths generally > 0.5 mm. This is only suitable for environment A1 (internal) and where the concrete slab will always have a floor covering in service.

(E) Additional requirements for localised areas

(E.1) Additional reinforcement will be required where the slab is subject to localised high tensile stresses due to deformation compatibility. Such locations are:
(a) Where secondary beams frame into or over primary beams;
(b) Where beams frame into columns;
(c) Where slabs frame into shear walls and other rigid elements;
(d) Guidance on these is given in HERA Report R4-113, *Notes Prepared for a Seminar on Composite Design and Construction*, 2002.

(E.2) Additional reinforcement will be required around openings in floors and re-entrant corners, comprising additional bars placed diagonally to the corner and anchored in accordance with NZS 3101.

(E.3) Slab depth to account for all reinforcement. Designers shall ensure that allowance is made within the depth of the slab and especially the depth of the cover slab (t_o-h_{rc}) for all covers and reinforcement including lapping (including corner lapping of mesh), trimmer and drag bars (to transfer diaphragm action or for dependable inelastic response in fire).

(Amendment No.2, October 2007)

Add the following new subclause after **C13.2.3**: (page 221)

C13.2.4 Bases for design and construction

When the floor system is designed, decisions will be made on the extent of propping and precambering to be used and the method for placing the concrete to achieve the required surface finish. These decisions have important implications for deflection of the floor system under the wet concrete, loads on the floor system during construction and long-term deflections of the composite floor system.

If changes are proposed to these design bases prior to construction, then the implications of these on the adequacy of the floor system during construction and in service must be determined and changes made to the design where required. See HERA Report R4-107 (13.50) for details. In general:

(a) Changing from propped to unpropped construction or from precambered to non-precambered beams will increase the depth of concrete placed and may reduce the unsupported deck spans.

(b) Changing from unpropped to propped construction may increase the long-term deflection of the composite floor system, may lead to increased crack widths and will influence the construction programme.

A method for calculating the increased concrete loading from ponding is given in section 1.3 of (13.50). Alternatively the specified thickness of the concrete over the floor system may be increased by $0.7\Delta_m$, where Δ_m is required from 13.2.4.3.

(Amendment No.2, October 2007)

C13.3.2.1 Nominal shear capacity of connectors (page 222)

Add the following material to the end of the subclause:

The replacement equation for shear stud capacity is a simplification of Equation 2.37 from (13.46) which provides better agreement with experimental tests, e.g. (13.47) and removes the artificial representation of shear stud failure as occurring either within the concrete or within the stud. That equation is given below. The simplification involves substituting for the elastic moduli of steel and concrete and solving for n = 5, which is a minimum realistic number of studs in the shear span.

$$q_r = \alpha_{dc}\left(4.3 - \frac{1.1}{\sqrt{n}}\right)A_{sc}f_u^{\,0.65}f_c^{\,0.35}\left(\frac{E_c}{E_s}\right)^{0.40} \leq 0.8\, f_u\, A_{sc} \quad \text{.....................................(Eq. C13.3.2.1)}$$

where

n = the number of studs in the shear span from Equation 13.4.9 (1) or n' from Equation 13.4.9(2), when the latter equation is required.

(Amendment No.2, October 2007)

C13.3.2.2 Stud shear connectors used with profiled steel deck (page 222)

Add the following paragraphs to the end of the subclause:

The new subclause (f) in 13.3.2.2.1 addresses the approach to take when through deck studs in a secondary beam configuration cannot be welded in the centre of the rib. This requires studs to be placed on the "weak" side or the "strong" side of the rib. Push off tests (13.47) show a difference in stud capacity of some 20 % between these two positions. However, full scale tests (13.49) show that provided the studs are placed alternately on the weak side and the strong side along the beam, the net result is the same as if all studs had been placed in the centre.

Compared with test results (13.47), the existing Equation 13.3.2.3 is significantly unconservative, especially for two studs. The replacement equation is from Eurocode 4 (13.48).

(Amendment No.2, October 2007)

C13.4.10 Longitudinal shear (page 228)

Add the following paragraphs to the end of the subclause:

The requirements to suppress a longitudinal shear failure in the cover slab have been recognised in past editions of the Standard and are not changed in this amendment.

However where the concrete rib is relatively narrow the shear stud strength is limited by cracking along the line of the studs. When this happens it is important that the post splitting capacity does not abruptly reduce. To prevent this, transverse reinforcement placed in specific locations is required and the relevant provisions are given in the new subclause 13.4.10.4. They are based on (13.46), which provides guidance on the appropriate placement of this reinforcement.

(Amendment No.2, October 2007)

C13.5 DESIGN OF COMPOSITE BEAMS WITHOUT SHEAR CONNECTIONS (page 229)

Add the following paragraph to the end of the clause:

These requirements are to change specific clause numbers referenced from NZS 3101:1995 to the relevant clause from NZS 3101:2006.

(Amendment No.2, October 2007)

C13.8.1 Scope (page 230)

Add the following paragraph to the end of the subclause:

The Amendment No. 2 requirements are to change specific clause numbers referenced from NZS 3101:1995 to the relevant clause from NZS 3101:2006.

(Amendment No.2, October 2007)

C13.8.2.3, C13.8.2.4 Longitudinal and transverse reinforcement requirements (page 231)

Change the last two sentences of the second paragraph to read:

The 1982 edition was revised into limit state format as NZS 3101:1995 and then NZS 3101:2006. Changes to the clause numbers in these editions of NZS 3101 have necessitated changes in the cross referencing from this Standard.

(Amendment No.2, October 2007)

REFERENCES TO SECTION C13 (page 232)

Alterations to references are as follows:

Change the following references:
13.8 from NZS 4203 to AS/NZS 1170 set Structural design actions
13.13 from NZS 3101:1995 Part 1 to NZS 3101:2006 Part 1
13.14 from NZS 3101:1995 Part 2 to NZS 3101:2006 Part 2
13.19 from the 1992 edition to the 1996 edition

Add the following new references to page 234:

13.47 Gillies, A.G. Clifton, G.C. Zaki, R. and Butterworth, J Recommendations for Shear Connector Design and Detailing for Interior and Exterior Secondary Composite Beams Proceedings of the 2005 Australasian Structural Engineering Conference.

13.48 EN 1994-1-1:2004 Eurocode 4: Design of Composite Steel and Concrete Structures – Part 1-1: General Rules and Rules for Buildings, European Committee for Standardization, Brussels, Belgium.

13.49 Hicks, S.J. Performance of Through-deck Welded Stud Connectors in Full-scale Composite Beams, Proceedings of the 2007 Pacific Structural Steel Conference.

13.50 Clifton, G.C. and El Sarraf (Zaki) R. Composite Floor Construction Handbook HERA, Manukau City, New Zealand, HERA Report R4-107, 2005.

13.51 Fussell, A. Xiao, H. Hyland C.W.K. STEELDECK Design Guide for Concentrated Loads on Metal Deck Slabs, SCNZ -11 Amd 1, Steel Construction New Zealand Inc., Manukau City, December, 2006.

(Amendment No.2, October 2007)

C15.3.4 Column slice (page 253)

Add the following paragraph to the end of the subclause:

The new subclause (d) is based on the 2005 AISC specification and allows relaxation of overly conservative requirements which impact on erection practice.

(Amendment No.2, October 2007)

Appendix CC CORROSION PROTECTION (page 265)

Add the following new paragraph:

Incorporates reference to important new document on coatings specification for corrosion and also addresses a common area of poor performance.

(Amendment No.2, October 2007)

Appendix CH ELASTIC RESISTANCE TO LATERAL BUCKLING (page 272)

Add the following new sentence after the **Appendix CH** heading and before **CH1 General**:

Change of status reflects the importance of the provisions to good performance.

(Amendment No.2, October 2007)

Appendix CJ (page 274)

Add the following paragraph immediately under the title:

The change of status from informative to normative reflects the importance of the provisions to good performance and corrects an error in one of the equations.

(Amendment No.2, October 2007)

Appendix CM DESIGN PROCEDURE FOR BOLTED MOMENT - RESISTING ENDPLATE CONNECTIONS (page 276)

Delete all of the the existing commentary and **replace** with:
Appendix M is deleted because it is too narrow in scope and contains errors. A design procedure for these connections is given in the SCI Publication P207:1995 *Joints in Steel Construction: Moment Connections*, published by the Steel Construction Institute, Ascot, England. A wide range of moment endplate connections are given in HERA Report R4-100:2004, *Structural Steelwork Connections Guide* and design of the columns for welded moment and moment endplate connections is covered by HERA Report R4-142, *Eccentric Cleats in Compression and Columns in Moment-resisting Connections*, Manukau City, New Zealand, 2007.

(Amendment No.2, October 2007)