NZS 3404:Parts 1 & 2 :1997

STEEL STRUCTURES STANDARD

AMENDMENT No. 1

June 2001

REVISED TEXT

EXPLANATORY NOTE

Amendment No. 1 to Parts 1 and 2 incorporates technical and editorial changes and includes items and references by way of clarification.

APPROVAL

Amendment No. 1 was approved on 27 October 2000 by the Standards Council to be an amendment to NZS 3404: Parts 1 and 2:1997.

Instructions for the use of Amendment No. 1

This Amendment has been published in two different ways for each Part.

Firstly, the Amendment consists of "Minor changes, Deletions and Insertions" which are to be made by hand on the user's copy.

Secondly, text and tables have been provided in a pre-printed perforated format which are to be inserted into the Parts.

Holders of NZS 3404:Parts 1 and 2 are advised to make the minor alterations by hand, cut out the perforated amendments and glue or tape them into the Standard as per the instructions given.

That this has been done should then be noted under "AMENDMENTS" on the inside front cover of the document.

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June 2001

Attn : All Steel Structures Standard Users

Another (Steel) Industry First !!

Following on from the well received, industry subsidised, NZS 3404 Steel Structures Standard reprint/revision in 1997, the Steel Industry is leading the way with another first – a free to users, easy to apply, amendment.

Why ? Anecdotal evidence is that only a proportion of all materials standards users actually purchase a regular amendment, and that of these, only a small proportion fully incorporate these into their design standard. The consequence is that a majority of designers are using an out-of-date document. This initiative is aimed to substantially increase the efficacy of this process.

How ? By use of an innovative, two pronged initiative :

i) By **pre-funding** the amendment, so that all steel standard users receive the amendment free of charge, and

ii) By trying out a new '**easy to implement**' format. In addition to a revised layout providing concise insertion instructions and separate technical content, the amendment is being provided on pre-perforated pages – aimed at minimising the user effort required to incorporate into their design standard.

Industry funding for this initiative has been provided in two parts :

i) Pre-funding of the SNZ amendment costs was made possible by the contributions of :
- Asmuss
- BHP New Zealand Steel
- Fletcher EasySteel
- HERA
- Steel & Tube
- Vulcan

ii) Funding of the printed, perforated pages was by SCI-NZ, while the editing & re-formatting work was by BHP NZ Steel in conjunction with Standards New Zealand.

SCI-NZ is a national association of steel industry participants, including the above contributors, and including manufacturers, merchants, fabricators, and other allied industry participants.

Please find the enclosed amendment, for incorporation in to your Steel Structures Standard. We welcome comment on this initiative, or any other enquiries about SCI-NZ, to bird.geoff.gd@bhp.com.au., or call 0800 Ph SCI NZ (0800 747 246).

Yours faithfully
Geoff Bird

Chairman
SCI NZ

SCI-NZ
STEEL CONSTRUCTION INDUSTRY NEW ZEALAND

NZS 3404 1997:Part 1:1997 Amendment No. 1, June 2001

Minor Changes, Deletions, and Insertions – to be done by hand

Page No., **Clause**	*The content on this page covers amendment items of a relatively minor nature that are most easily effected by manual means.* *Please change, delete, insert, or replace each item as per the instructions for that item.*
p. 24, **1.4**	**Change** "A_{rv}" to "A_{rt}".
p. 27, **1.4**	**Delete** the definition for "e_c, e_t ".
p. 46, **2.2.3**	In line 4 **replace** "BS 18" **with** "BS EN 10002-1".
p. 55, **3.3 (c)**	**Append** ", or clause 11.5".
p. 149, **8.4.2.2.1**	After the equation in line 8, **delete** "≤ M_{rx} or M_{ry} as appropriate".
p. 170, **9.5.4**	**Delete** this clause, which is now replaced by the amended 9.5.2.
p. 198, **Table 10.5.1(2)**	In Detail category 36, **change** the accompanying text to read: "t_r and t_p >25 mm".
p. 218, **Table 12.2.6**	**Change** the following minimum member ductility categories : • Structural ductility category 2, with capacity design, secondary member; from "3(4) (see Note 2)" to "2(4) (see Note 2)". • Structural ductility category 3, with capacity design, secondary member; from "4" to "3(4) (see Note 2)".
p. 219, **Table 12.2.8(2)**	Under the row headed "Steel grade" **change** the first two instances of "ex Aust" **to** "from A/NZ".
p. 228, **Table 12.5**	In NOTE 5, line 2, **change** the word "load" to "force".
p. 229, **12.6.2.1**	Definitions to Eq. 12.6.2 : • **Change** definition of "M_r^*" to "M_{res}^*". • In the definition for M_s, **change** "M_{sp} or M_{pr}" to "M_s or M_r".
p. 268, **Table 13.1.2(1)**	**Change** the values of ϕ_{sc} for shear connectors given, as follows: • "situated in positive moment regions", **change** the existing "0.80" to "1.0". • "situated in negative moment regions", **change** the existing "0.60" to "0.75".
p. 273, **13.3.2.2.2**	• In (a), at the end of Eq. 13.3.2.3, **change** "≥ 1.0" to "≤ 1.0" . • In (b), at the end of Eq. 13.3.2.5, **change** "b_r/h_r" to "b_r/h_{rc}".
p. 294, **14.3.5.2.3 (c)**	**Change** the value "1.5 d_f" to "2.5 d_f" .
p. 319, **Appendices** Contents	• Appendix C, end of title, **change** the word "informative" to "normative". • Appendix M, end of title, **change** the word "informative" to "normative".
p. 323, **Appendix A,** "Other Publications"	**Delete** reference to PD 6493-1991 *(Replaced with BS 7910 reference on p.322)*
p. 328, **Appendix C7**	• **Change** "AS 1629.9" to "AS 1627.9". • **Change** "AS 1629.10" to "AS 1627.10".

Page No., **Clause**	*Please change, delete, insert, or replace each item as per the instructions for that item.*
p. 330, **Appendix C7**	In the title for BS EN 22063:1994, **change** the word "sparying" to "spraying".
p. 352, **Appendix M, Figure M1**	In the definitions of variables for Figure M1, **change** "$a \approx 2.5\ d_{fb}$" to "$a \leq 2.5\ d_{fb}$".
p. 356 & 357, **Appendix M3**	• In Eq. M2.9.1 **change** the term "f_{yep}" to "ϕf_{yep}". • In Eq. M2.9.2 **change** the term "f_{yep}" to "$\phi\ f_{yep}$". (Note : the ø factor goes inside the brackets)
p. 366, **Appendix P, Table P3.3**	In the column underneath the heading "Clause", **change** "N" to "P" 14 times.
p. 381, **Index**	**Delete** the following items from the index. Action effect .... 4.1 design .... 3.3, 3.4, 4 Analysis advanced 4.1.1, 8.2

Page No., **Clause**	*The following content has been provided in a pre-printed perforated format, in order to assist direct insertion into NZS 3404:Part 1:1997 with minimal effort.* *Please tear off the perforated amendment item contained in the right hand column (below), place it into the Standard as per the location and instructions contained in the left hand column adjacent to each item.*
p. 41, **1.4** **Add** definition	ϕ_{fire} = strength reduction factor for fire emergency conditions (11.5)
p. 85, **Table 5.2** **Append** new notes	(6a) These limits are applicable to the webs of rectangular and square hollow section members. (6b) These limits are applicable to the webs of members other than rectangular and square hollow section members.
p. 88, **5.3.1.2** **Append** note after item (b)	NOTE – Typically, 5.3.1.2(b) will only be applied to a free-standing cantilever member with one end unrestrained.
p. 91, **5.4.2.2** **Replace** the whole clause	**5.4.2.2 Partial section restraint (P)** A cross section of a member may be considered to be partially restrained (P) if the restraint or support effectively prevents lateral deflection of some point in the cross section other than the critical flange, and provides partial restraint against twist rotation of the section, as for example in figure 5.4.2.2. A cross section of a member may also be considered to be partially restrained (P) if twist rotation of the cross section is effectively restrained and lateral deflection of the cross section is partially restrained. NOTE – When the bolt hole size is larger than the nominal diameter specified in 14.3.5.2.1, tensioned property class 8.8 bolts (category 8.8/TB) to 15.2.4 and 15.2.5 shall be used in any bolted connection providing partial restraint between the restrained and restraining members, unless the effect of bolt slip on the restraint is accounted for by the use of Appendix H.

p. 102, Table 5.6.3(2) Notes : Append note (2) to table

(2) For singly-symmetric sections loaded through the centroid perpendicular to the axis of symmetry (e.g. channel sections subject to major axis bending), any reduction in member moment capacity arising from the offset of the shear centre from the centroid along the symmetry axis is accounted for through the load height factor, when the load itself or the structural system transferring the load to the segment is laterally restrained. When either the load or the transferring structural system is laterally unrestrained, then the member must be designed for the torsion arising from the offset of the shear centre and centroid, in addition to the direct bending.

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p. 108, **5.11.2.3 Replace** heading

5.11.2.3 Shear capacity of a slender web with uniform shear stress distribution

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p. 124, **6.2.4.1 Add** note after paragraph

NOTE – For circular hollow sections, the effective inside diameter equals the effective outside diameter minus twice the actual wall thickness.

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p.144, **8.1.4 Replace** the whole clause

8.1.4 Significant axial force

For application of section 8, the level of design axial force (N^*) shall be considered significant unless it complies with one of (a), (b) or (c) below:

(a) For a member subject to axial compression and either :

(i) Major principal x-axis bending with full lateral restraint in accordance with 8.1.2.3:

$N^* \leq 0.05\, \phi N_s$ if the member is an I-section or channel section

$N^* \leq 0.05\, \phi N_{cx}$ for all other sections

or

(ii) Bending in the major principal x-axis or the minor principal y-axis, for all sections except as covered by(c) :

$N^* \leq 0.05\, \phi N_{cy}$

(b) For a member subject to tension and bending in the major principal x-axis or the minor principal y-axis

$N^* \leq 0.05\, \phi N_t$

(c) For an angle to 8.4.6:

$N^* \leq 0.05 \phi N_{ch}$

where

N_{ch} is as defined in 8.4.6.1

N_{cx} is as defined in 8.4.2.2.1 for buckling about the major principal x-axis

N_{cy} is as defined in 8.4.4.1.1

N_s is as defined in 8.3.1

N_t is as defined in 8.3.1.

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p. 153, **8.4.4.1.2 Replace** definition

M_{bxo} = the nominal member moment capacity of the segment or member calculated using $\alpha_m = 1.0$ in accordance with 5.6

---

p. 153, **8.4.4.1.2 Replace** definition

L_z = the distance between restraints which effectively prevents twist of the section about its centroid (i.e. full or partial restraints).

---

p. 154, **8.4.5.1 Replace** heading & next two lines

8.4.5.1 Compression members or members subject to zero axial force

A member subject to a design axial compressive force (N^*) or to $N^* = 0$, and design bending moments M_x^* and M_y^* about the major x- and minor y- principal axes respectively shall satisfy the following :

p.169, **9.5.2**
Replace the whole clause

9.5.2 Pin and ply in bearing

A pin and a ply subject to a design bearing force $\left(V_b^*\right)$ shall satisfy:

$$V_b^* \leq \phi V_b$$

where

ϕ = strength reduction factor (see table 3.3)

V_b = nominal bearing capacity of the pin or ply, whichever is the least.

The nominal bearing capacity (V_b) shall be calculated as follows:

$$V_b = 1.4\, f_{yb}\, d_f t_p k_p$$

where

f_{yb} = yield stress of the pin or ply, whichever is the least

d_f = pin diameter

t_p = connecting plate thickness(es)

k_p = 1.0 for pins without rotation, or
= 0.5 for pins with rotation.

p.208, **11.5**
Replace the whole clause

11.5 DETERMINATION OF LIMITING STEEL TEMPERATURE

The limiting steel temperature (T_l) shall be calculated as follows:

$$T_l = 905 - 690 r_f$$

where r_f is the ratio of the design action on the member under the design load for fire specified in NZS 4203 to the design capacity of the member ($\phi_{fire}\, R_u$) at room temperature, for fire emergency conditions. The strength reduction factor for fire emergency conditions, $\phi_{fire} = (\phi/0.85) \leq 1.0$, where ϕ is given by table 3.3(1) or table 13.1.2(1), as appropriate.

p. 228, **Table 12.5**
Append new note

(6) These limits are applicable to the webs of rectangular and square hollow section members.

p.229, **12.6.1.3**
Insert the following new clause :

12.6.1.3 Design bending moment for application for restraint to category 1, 2 and 3 members

When calculating the design moment magnitude and distribution along the member for application of clause 12.6.2;

(a) At the point of maximum moment in each yielding region, set the design bending moment for calculation of restraint equal to the design section moment capacity, reduced by axial force where appropriate, i.e.

$$M^*_{res,max} = \phi M_s$$

(b) At each point along the member,

$$M^*_{res} = M^* \left(\frac{\phi M_s}{M^*_{max}}\right) \quad \text{.........(Eq. 12.6.1)}$$

where

M^*_{res} = design bending moment for calculation of restraint to clause 12.6.2
M^* = design bending moment from analysis at the point under consideration
M^*_{max} = maximum design bending moment from analysis at the adjacent yielding region(s)
ϕM_s = ϕM_s or ϕM_r as appropriate (see 1.4).

(c) Use the value of M^*_{res} from Equation 12.6.1 in the application of clauses 12.6.2.1 to 12.6.2.5.

p.230, **12.6.2.2**
Delete the whole clause and **replace** with:

12.6.2.2 Restraint of segments which contain one or more yielding regions
Each segment containing one or more yielding regions shall have $(\alpha_m \alpha_s) \geq 1.0$, where:

α_m and α_s are determined in accordance with 5.6, but $\alpha_m \leq 1.75$ for application of this clause.

p.230, **12.6.2.3.2**
Delete the whole clause and **replace** with:

12.6.2.3.2
The type of restraint required within the yielding region shall be as specified in table 12.6.3.

For category 1 and 2 members, at least one restraint of this type shall be provided within or at one end of each length of yielding region.

For category 3 members, at least one F restraint and as many other F or P restraints, as required, shall be provided within, or at one end of each length of yielding region.

NOTE – Where the yielding region is at the end of a member, the end connection will provide a suitable restraint to the yielding region. In most applications, this will be the only restraint required from 12.6.2.3.

p. 231, **12.6.2.5**
Replace item (a)

(a) When the member is subject to bending, these segments shall satisfy $\phi M_{bx} \geq M^*_{res}$ (see 5.6 for ϕM_{bx} and Equation 12.6.1 for M^*_{res}) in order that the required inelastic action is able to be developed in the yielding region or regions within the member.

p.235, **12.8.3.3**
Replace the whole clause

12.8.3.3 Limit on transverse loading on category 1, 2 and 3 columns
Except as noted in the following paragraph, direct transverse loading between the supports/ restraints of category 1, 2 and 3 columns shall be limited to that which generates a design moment less than or equal to 10 % of the design section moment capacity (ϕM_s) of the member about the appropriate principal axis or axes.

If, in a member subject to direct transverse loading, the position and pattern of the yielding regions is dependably unchanged by the application of the transverse load, then this limitation does not apply.

p.236, **12.8.4**
Replace items (b) and (c) with :

(b) *One seismic-resisting system is category 1, the other is category 2*

(i) The seismic-resisting systems shall be designed for seismic forces applied separately along each principal direction, using capacity design for the category 1 system and, when appropriate, for the category 2 system; then

(ii) The design actions from (b)(i) shall be considered acting concurrently on the column; and

(iii) Column design for concurrent action shall be in accordance with the requirements of sections 5 - 8 and 13, with the design capacity (ϕR_u) being not less than 0.67 times the concurrently acting design actions (S^*).

(c) *Both seismic-resisting systems are category 2*

(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. For CBFs, seismic design actions shall incorporate the C_s factor from 12.12.3.1 or 12.12.6.3. For structures with seismic-resisting systems located along 2 perpendicular directions, the direction of action shall be 45° to each system; and

(iii) The design capacity of the column shall be not less than:

(A) 0.80 times the concurrently acting design actions (S^*) if the column satisfies the requirements of 12.4, 12.5 and 12.6 for a category 2 member; or

(B) 1.0 times the concurrently acting design actions (S^*) if the column satisfies the requirements of 12.4, 12.5 and 12.6 for a category 3 member.

p.239, **12.9.2.2.1** **Replace** item (2) with:	(2) The combination of: (i) 50 % of the design section moment capacity, reduced by N_g^*, of the smaller member ($0.5 \phi M_r$); and (ii) 25 % of the design shear capacity of the smaller member ($0.25\ \phi\ V_v$).
p.239, **12.9.2.2.2** **Replace** item (2) with:	(2) The combination of: (i) 30 % of the design section moment capacity, reduced by N_g^*, of the smaller member ($0.3 \phi M_r$); and (ii) 15 % of the design shear capacity of the smaller member ($0.15\ \phi\ V_v$). where N_g^* = design axial force generated by gravity loading alone (e.g. dead, live loading).
p. 272, **13.3.2.1** **Replace** equation	$q_r = 0.4\ \alpha_{dc}\ A_{sc}\sqrt{(f_c' E_c)} \quad \leq \quad 0.8\ f_u\ A_{sc}\ \alpha_{dc}$ .... (Eq.13.3.2.1)
p. 322, **App. A**, "British Stds", **Add** reference	BS 7910 — Guide on methods for assessing the acceptability of flaws in fusion welded structures
p. 328, **C6** **Replace** the last line in the 2nd para. with:	shall be restored by a suitable zinc paint conforming to AS/NZS 3750.9 or to AS/NZS 3750.15, or with thermal zinc spray.
p. 353, **M2.1.2** **Replace** clause	**M2.1.2** The design tension capacity of the bolt group at the beam tension flange, $\sum(\phi N_{tf})$ shall be greater than the lesser of the unstiffened column flange design capacity, ϕN_m, or the design tension capacity of the endplate, calculated from M3 once the endplate thickness is determined.
p. 355, **M2.2.3** **Replace** clause	**M2.2.3** The design tension capacity of the bolt group at the beam tension flange, $\sum(\phi N_{tf})$ shall be greater than the lesser of the stiffened column flange design capacity, ϕN_{ms}, or the design tension capacity of the endplate, calculated from M3 once the endplate thickness is determined.
p. 356, **M3 (a)** **Replace** title	(a) **When the endplate is thinner or equal to the column flange (plus backing plate, if present):**
p. 381, **Index** **Add** item	Alternative design provisions 8.1.5
p. 383, **Index** **Add** item	Compact sections .... 5.2.3 definition 1.3
p. 390, **Index** **Add** item	Plastic analysis .... 4.6 limitations 4.6.2

p. 50, **Table 2.6.4.1**
Replace table

Table 2.6.4.1 - Permissible service temperatures according to steel type and thickness

Steel type (see table 2.6.4.4)	Permissible service temperatures (°C)					
	Thickness (mm)					
	0 5	10	20	30	40 50	≥70
1	–20	–10	0			+5
2	–30	–20	–10		0	
3	–40	–30	–20	–10		
4	–10	0	0	0		+5
5	–30	–20	–10	0		
6	–40	–30	–20	–10		
7A	–10	0				
7B	–30	–20	–10	0		

p. 52, **Table 2.6.4.4**
Replace table

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
1	C250	HA200 HA250, HU250 HA300, HU300 HA300/1	200 250 300	250 300	S275 S275JR	SM 400A
2	C250L0	–	–	250L0 300L0	S275JO	SM 400B
3	–	–	250L15 300L15	250L15 300L5	S275J2G3/ S275J2G4	SM 400C
4	C350	HA350 HA400	350 WR350	350	S355 S355JR	SM 490YA
5	C350L0	–	WR350L0	350L0	S355JO	SM 490YB SM 520B
6	–	–	350L15 400L15	350L15	S355J2G3/ S355J2G4	SM 520C
7A	C450	–	–	–	–	–
7B	C450L0	–	–	–	–	–

p. 85, **Table 5.2**
Replace case number 4 with:

4	Flat (Compression at one edge, tension at the other)	Both	Any Any Any	45[6A] 82[6B] –	60[6A] 130[6B] 180[5]	–

p. 228, **Table 12.5**
Replace case numbers 3, 4 & 5 with:

3	Flat	Both	SR HR LW, CF HW	25 25 25 25	30 30 30 30	40 40 40 35	60[3] 60[3] 60[3] 60[3]
	(Uniform compression)						
4	Flat	Both	Any	82[5] 30[6]	82[5] 40[5,6]	101[4,5] 55[5,6]	161[4] 75[6]
	(Either non-uniform compression or compression at one edge, tension at the other)						
5	Circular hollow sections		SR HR, CF LW HW	35 35 30 30	50 50 42 42	65 65 60 60	170[3] 170[3] 170[3] 170[3]

p. 301, **Table 14.4.5**
Replace the whole table

Table 14.4.5 – Tolerance on camber and sweep in beams

Nominal size	Sweep	Camber
I-sections with a flange width less than 150 mm	$\frac{\text{Length}}{500}$	$\frac{\text{Length}}{1000}$
I-sections with a flange width approximately equal to the depth		
Lengths of 14 m and under	$\frac{\text{Length}}{1000}$ but not more than 10 mm	
Length over 14 m	$10\text{ mm} + \frac{\text{Length (mm)} - 14\,000}{1000}$	
All other I-sections	$\frac{\text{Length}}{1000}$	$\frac{\text{Length}}{1000}$

NOTE –
(1) For I-sections with a specified precamber, camber shall be +0, −10 mm.
(2) Owing to the extreme variation in the elastic flexibility of I-sections about the Y-Y axis, difficulty may be experienced in obtaining reproducible sweep measurements. Appendix C of AS/NZS 3679:Part 1 provides a means for measuring sweep.

p. 378, **Table Q5**
Replace the whole table

Table Q5 – Corresponding tables

AS 4100 table	Corresponding NZS 3404 table
4.6.3.4	4.8.3.4
5.2	5.2
5.6.1	5.6.1
5.6.2	5.6.2
5.6.3(1)	5.6.3(1)
5.6.3(2)	5.6.3(2)
5.6.3(3)	5.6.3(3)

AS 4100 table	Corresponding NZS 3404 table
6.2.4	6.2.4
6.3.3(1) & 6.3.3(2)	6.3.3(1)
6.3.3(3)	6.3.3(2)
7.3.2	7.3.2
10.4.1	2.6.4.1
10.4.4	2.6.4.4

Minor Changes, Deletions, and Insertions – to be done by hand

Page No., **Clause**	*The content on this page covers amendment items of a relatively minor nature that are most easily effected by manual means.* *Please change, delete, insert, or replace each item as per the instructions for that item.*
p. 59, **C5**	In item (1) **insert** "ϕ" before "f_y", to read "$\phi M_{sx} = \phi f_y Z_{ex}$ from 5.2.1".
p. 60, **C5, (2.2)**	In item (2.2) **delete** the last sentence.
p. 60, **C5, (2.3)**	In item (2.3), line 2, **delete** the words "or Appendix A5 of (5.36)". In item (2.3), paragraph 2, line 2, **delete** "or 5.36".
p. 68, **C5.4.2**	In paragraph 2, line 1, **delete** the words "or Appendix A5 of (5.36)". In paragraph 2, line 2, **replace** "5.4.7.2 of (5.36)" with "2.5 of (5.40)".
p. 69, **C5.4.2.2**	In paragraph 2, line 1, before the word "purlin" **insert** the word "steel". In paragraph 2, line 5, before the word "purlin" **insert** the word "steel".
p. 111, **Figure C8.1.2**	In the fourth box down from the top, **delete** the word "load".
p. 181, **C12.4.1.2**	In the seventh line, **change** "0.75 kJ/m" to "0.75 kJ/mm".
p. 206, **C12.12.3**	In Equation C12.2.3(2), **change** "category 3A" to "category 3".
p. 264, **C17.5.2.1**	Step 3, line 6, **change** the word "minimun" to "minimum" in both places.

Page No., **Clause**	*The following content has been provided in a pre-printed perforated format, in order to assist direct insertion into NZS 3404:Part 2:1997 with minimal effort.* *Please tear off the perforated amendment item contained in the right hand column (below), place it into the Standard as per the location and instructions contained in the left hand column adjacent to each item.*
p. 23, **C2.6** **Append** new paragraph	The material designations for steel grades to BS EN 10025 presented in the 1997 edition of table 2.6.4.4 were interim designations published in BS EN 10025:1990. The final designations were published in the 1993 edition of the BS EN 10025 and are incorporated into this Amendment No. 1. Designations for steels from Britain have changed significantly since 1986; details of the new material designations and a comparison between these and the pre-1986 designations can be obtained from HERA.
p.31, **C3.3** **Replace** paragraph 4 on this page	For non-seismic applications, the design capacity of the member or connection is determined in accordance with 3.3(c). Note that the strength reduction factor (ϕ) is always less than one for the ultimate limit state, except as given by clause 11.5 for fire emergency conditions.
p.33, **C3.4.4** **Add** paragraph to end of clause	In 1997, the American Institute of Steel Construction published a revised joint American/Canadian design procedure for control of in-service floor vibration that is wider in scope than the procedure contained in Appendix B13 of (3.15). This publication is available through HERA; details of it are contained in an article on pp. 25-27 of the HERA *Steel Design and Construction Bulletin*, Issue No. 56, 2000.
p. 64, **C5.2.2** **Add** paragraph to end of clause	Rectangular and square hollow sections are much more susceptible to loss of bending strength from combined local flange and web buckling than is the case for I-sections. Because of this, their web slenderness, for a given flange slenderness, needs more stringent limits than are in table 5.2. Suitable limits for web slenderness on these types of cross section have been introduced in Amendment No. 1. A background to these limits is to be given in the HERA Steel Design and Construction Bulletin, Issue No. 55, April 2000.

p. 69, **C5.4.2.2** **Replace** the last three paragraphs of clause	If the beam being supported off the web side plate connection is carrying a rigid (e.g. concrete) floor slab, then full restraint (F) can be assumed for the top flange critical (see (5.40)). The comment relating to use of fully tensioned bolts into oversize holes in purlins highlighted in C5.4.2.1 also applies to partially restrained connections. A background to the second option for partial restraint (P), as introduced through Amendment No. 1, is given in the HERA *Steel Design and Construction Bulletin*, Issue No. 48, 1999, pp. 1, 2.
p. 82, **C5.10.7** **Add** paragraph to end of clause	The general design philosophy and procedure given in (5.39) has now been adapted for New Zealand application. Details are given on pages 1-6 and appendices A-C of the HERA *Steel Design and Construction Bulletin*, Issue No. 53, 1999.
p. 91, **C5** **Replace** reference 5.40	5.40 HERA. 1997. Restraint Classifications for Beam Member Moment Capacity Determination to NZS 3404:1997. HERA Report R4-92. Published by HERA, Manukau City.
p. 112, **C8.1.2** **Replace** last paragraph of clause	Clause 8.1.2.4 then provides the important requirement that a member which contains one or more yielding regions and is subject to combined bending moment and significant axial actions must have full lateral restraint.
p. 112 , **C8.1.4** **Add** paragraph to end of clause	A background to the Amendment No. 1 changes to clause 8.1.4 (a) is given on page 5 of the HERA *Steel Design and Construction Bulletin*, Issue No. 43, 1998. The changes described therein have been rewritten when putting them into the amendment to make them more compact and to avoid the need for a new sub-clause.
p. 116, **C8.4.4.1** **Delete** the last paragraph and **replace** with:	Note that L_z in 8.4.4.1.2 is the distance between partial or full restraints that prevent twist of the section about its centroid. Not all restraint conditions given in clause 5.4 do this; refer to section 4.6.2.2 of (8.14) for guidance on effective twist restraints. The changes introduced in Amendment No. 1 to the definitions for M_{bxo} and L_z are for clarity; the meanings are not changed.
p. 116, **C8.4.5.1** **Replace** heading and first paragraph	**C8.4.5.1 Compression members or members subject to zero axial force** This clause gives a power law approximation for the biaxial bending capacities of members with axial compression or for biaxial bending alone ($N^* = 0$). For the term (M_{cx}) associated with bending about the major principal *x*-axis, the answer depends on whether or not the member has full lateral restraint to 8.1.2.3.
p. 141, **C9.5.2** **Replace** last paragraph	The factor k_p of 0.5 for a pin that allows rotation reflects the fact that continual movement of the pin plates around the pin circumference creates a wearing effect. Amendment No. 1 now applies these provisions to the ply as well. See the new C9.5.4, introduced by this amendment, for the background to this change.
p. 141, **C9.5.4** **Replace** the whole clause	**C9.5.4 Ply in bearing** Pin plates are designed for bearing using the provisions of 9.3.2.4, as for a ply in bearing due to a bolt. The 1997 provisions treated the pin plates in bearing as for a ply in bearing due to a bolt. However, application of these to an experimental test rig component, in 1999, has shown them to be unconservative. The provisions introduced in Amendment No. 1 treat the pin plates in the same manner as the pin and have delivered good performance in the example quoted in the HERA *Steel Design and Construction Bulletin*, Issue No. 51, 1999, pp. 9-11, which presents the background to these changes. Because the pin plates are now treated the same as the pin, clause 9.5.2 has been modified to cover both pin and ply, with the original clause 9.5.4 being deleted.
p. 159, **C10.1.1** **Replace** the last paragraph with two paragraphs	The application of a fracture assessment in accordance with e.g. BS 7910:1999 offers an alternative to the *S-N* design approach presented in this clause or in (10.8). General guidance on all the aspects of design for fatigue resistance, including a comparison between the provisions of this Standard, BS 7608 (10.8) and other commonly used Standards is contained in HERA Report R8-19 *Seminar Notes on Fatigue in Welded Construction*, by Bayley, C. and Scholz, W., published by HERA in February 2000.

p. 167, **C11.5**
Add a 2nd paragraph

The use of a more lenient strength reduction factor for emergency conditions (ϕ_{fire} = 1.0 for most applications) reflects an accepted lower safety index for individual member performance under fully developed fire conditions. This modification, introduced by this Amendment, makes the approach for structural steel and composite steel/concrete members in fire consistent with that applied to reinforced concrete (see clause 3.6 of NZS 3101 (13.13)), timber (see section 6.9 of (11.21)) and composite slabs on profiled steel decking (see section 3.4 of (11.20)). The specification of $\phi_{fire} = (\phi/0.85)$ makes the reduction in factor of safety constant across all actions and components, consistent with the approach for earthquake embodied in the Ideal Capacity Factor (see clause 1.3).

p. 176, **C12.2.6**
Add paragraph to end of clause

Depending on the degree of foundation flexibility, the extent of ductility demand in secondary members whose position makes them susceptible to inelastic action (e.g. column bases with rigid connections into the foundations) may sometimes be greater than that allowed for in table 12.2.6. The two changes introduced in Amendment No. 1 will ensure that the ductility demand on the secondary member does not exceed the ductility capability of that member.

p. 182, **C12.5**
Add three paragraphs to end of clause

The Amendment No. 1 changes to web slenderness limits for rectangular and square hollow sections (case number 4 in table 12.5) are introduced because these sections are more susceptible to loss of bending strength from combined local flange and web buckling than is the case for I-sections. Because of this, their web slenderness, for a given flange slenderness, needs more stringent limits than were in table 12.5. A background to the proposed new limits is given in the HERA *Steel Design and Construction Bulletin*, Issue No. 55, April 2000.

The changes to slenderness limits of category 4 elements subject to uniform compression, both edges supported (case number 3 in Table 12.5) are to bring these limits into line with those for category 4 elements subject to uniform compression, one edge supported (case number 1 in table 12.5). The ratio of this category 4 limit to the yield limit is now very similar for both cases.

The changes to slenderness limits for category 1 circular hollow sections arise from USA research which shows the previous limits (also based on earlier USA research), to be unconservative.

p. 182, **C12.6**
Add new 3rd paragraph to end of clause

A background to the changes to 12.6 made in Amendment No. 1 is given in the HERA *Steel Design and Construction Bulletin*, Issue No. 51, 1999, pp. 11, 12. The moment for restraint determination is termed M^*_{res} to avoid confusion with M^*_r, which relates to the section moment capacity reduced by axial force.

p. 186, **C12.8.3.3**
Replace clause

C12.8.3.3 Limit on transverse loading on category 1, 2 and 3 columns
The design bending moment from transverse loading may always be calculated assuming simple end support conditions (i.e. no joint fixity) for checking 12.8.3.3. The reason for the current clause and background to the Amendment No. 1 addition is given in the HERA *Steel Design and Construction Bulletin*, Issue No. 45, 1998, pp. 7, 8.

p. 189, **C12.9.2**
Replace the last paragraph

Analyses undertaken, principally by MacRae (12.6), show that these requirements are generally satisfactory, except that the minimum design actions for splices in the lower half-height of columns of category 1 and 2 systems, as required by the 1992 edition, have been extended to apply over all levels of these systems, based on the recommendations of Clifton (12.52). N^*_g is constant during an earthquake and acts to potentially reduce ϕM_s, therefore it can and should be taken into account in design of column splices. This is introduced through Amendment No. 1.

p. 194, **C12.9.5.3.2**
Delete all the clause from sub-heading "Design and detailing recommendations ..." onwards and **replace** with:

Design and detailing recommendations for doubler plate(s)

Doubler plate(s) required from 12.9.5.3.2 should be designed in accordance with the recommendations given in the HERA *Steel Design and Construction Bulletin*, Issue No. 57, 2000. (The article in Issue No. 57 supersedes previous guidance in the DCB on doubler plate design). Some of the key points of those recommendations are:

1. Do not use doubler plates that are too thin. As an approximate guideline only, if $t_p \leq t_{wc}/2$ or 8 mm is required, use one plate only, with a limit on the thickness of any one doubler plate of $t_p \leq t_{wc}$.

2. If a web side plate cleat frames into the doubler plate, ensure the design shear from the design gravity load on the web side plate cleat can be resisted. This shear does not need to be considered concurrently with the earthquake induced shears from the moment-resisting connection. The doubler plate to column welds must also be checked for this shear.

3. A minimum doubler plate thickness of 5 mm should be used.

4. The doubler plate is sized to fit within the panel zone region once the tension/compression stiffeners are in position. (In any practical joint detail, tension/compression stiffeners will always be required when doubler plate(s) are needed). The fitting of the doubler plate and sizing of the welds between it and the tension/compression stiffeners and between the tension/compression stiffeners and the column web are covered in HERA *Steel Design and Construction Bulletin*, Issue No. 57, 2000.

p. 217, **C13.1.2.1**
Add new 4th paragraph at bottom of page

The product $\phi_{sc}q_r$ is not changed by Amendment No. 1, as described in the amendments to C13.3.2.1.

p. 222, **C13.3.2.1**
Delete the last three paragraphs and **replace** with:

The design capacity, $\phi_{sc}q_r$, of shear connectors given by Equations 13.3.2.1 and 13.3.2.2 has not been altered by Amendment No. 1. What has been done is to multiply ϕ_{sc} by (1/0.8) and to multiply q_r by 0.8. This brings the factor of 0.8 directly into the equation for determining shear stud nominal capacity, which provides better agreement with experimental results. For more details, see pages 25 and 26 of the HERA *Steel Design and Construction Bulletin*, Issue No. 55, 2000. The experimental work on which these changes are based has been undertaken on end welded studs, represented by Equation 13.3.2.1, with the concepts also applied to Equation 13.3.2.2, which covers channel connectors.

The lower value of ϕ given for shear connectors in table 13.1.2(1) which are situated in negative moment regions takes account of the reduced holding power of the stud in cracked concrete, as determined by Johnson et al (13.26) through experimental testing. It is incorporated into current UK design practice (13.9, 13.30).

Equation 13.3.2.1 is only applicable for studs with a ratio of length/diameter (h_{sc}/d_{sc}) ≥ 4.0. Equations applicable to shorter studs, down to h_{sc}/d_{sc} = 3.0, are available, e.g. (13.25) and can be used. Alternatively equation 13.3.2.1 can be used if a 25 % reduction in stud strength determined from the first part of the equation (associated with concrete failure) is applied for studs with h_{sc}/d_{sc} = 3.0, with linear interpolation used for studs with values of h_{sc}/d_{sc} between 3.0 and 4.0. No reduction need be made in the second part of equation 13.3.2.1 (associated with failure of the stud itself) for studs with values of h_{sc}/d_{sc} between 3.0 and 4.0.

The role of the shear connector is to provide sufficient strength and stiffness in resisting longitudinal shear at the steel/concrete interface, so that:

(i) The shear stud will yield prior to splitting of the concrete rib occurring;

(ii) There will not be an abrupt or significant loss of shear capacity when splitting occurs, but a gradual decline from the peak shear capacity with increasing longitudinal slip.

These performance requirements will need to be determined by experimental test and/or rational analysis for shear connectors not covered by the provisions of (a) or (b) or outside the scope of 13.3.2.2 and 13.3.2.3. The concepts involved are described in detail in the HERA *Steel Design and Construction Bulletin* Issue No. 55, 2000, pp. 18-28, with the background to these concepts given in (13.46).

p. 228, **C13.4.10** **Add** new 5th paragraph	The design for longitudinal shear resistance involves a check for the adequacy to resist splitting of the concrete across potential longitudinal shear planes, such as those shown in figure C13.4.10. This form of splitting must not be confused with potential splitting of the concrete along the line of the shear connectors, due to high transverse tensile forces developed locally in the concrete near the bases of the connectors. The latter is an important factor to consider in the calculation of shear connector capacity; see 13.3.2. Detailed coverage of both forms of splitting is given in (13.46).
p. 234, **C13** **Replace** reference 13.42	13.42 Hyland, C. and Clifton, G.C. 1997. Deflection of Composite Floor Systems. HERA. Design and Construction Bulletin, No. 33. Also see Minor Revisions to Recommended Deflection Limits for Composite Floor Systems presented in HERA *Steel Design and Construction Bulletin*, Issue No. 52, p.9.
p. 234, **C13** **Append** new reference	13.46 Oehlers, D.J. and Bradford, M.A. 1995. Composite Steel and Concrete Structural Members Fundamental Behaviour. Elsevier Science Ltd., London, England.
p. 244, **C14.4.5** **Add** paragraph to end of clause	Experience has shown that the tolerances for camber given in table 14.4.5 are not always sufficiently stringent for I-section floor beams for which a specified precamber is given. An additional item has been added to rectify this.
p. 245, **C14.6** **Replace** last paragraph of clause	The HERA Specification (14.15) contains a more comprehensive check list for fabrication and erection, and AS/NZS 1554.1 contains a check list of contractual matters related exclusively to welding. The HERA Specification (14.15) also references articles covering the frequency of inspection for welded connections, bolted connections and welded shear studs.
p. 246, **C14 references** **Replace** reference 14.15	14.15 HERA. 1998. HERA Specification for the Fabrication, Erection and Surface Treatment of Structural Steelwork. HERA Report R4-99. Published by HERA, Manukau City.
p. 253, **C15.4** **Add** new 3rd paragraph at bottom of page	Guidance on the scope and frequency of inspection of bolts in bolted connections is presented in the HERA *Steel Design and Construction Bulletin*, Issue No. 46, 1998, pp. 8-10.
p. 255, **C15 references** **Replace** reference 15.7	15.7 HERA. 1998. HERA Specification for the Fabrication, Erection and Surface Treatment of Structural Steelwork. HERA Report R4-99. Published by HERA, Manukau City.
p. 259, **C16 references** **Replace** reference 16.2	16.2 HERA. 1998. HERA Specification for the Fabrication, Erection and Surface Treatment of Structural Steelwork. HERA Report R4-99. Published by HERA, Manukau City.
p. 266, App. **CD** **Append** paragraph at end	Recommendations on applying table D1 in practice are given in the HERA *Steel Design and Construction Bulletin*, Issue No. 44, 1998, pp. 2, 3.
p. 276, **CM1.1** **Add** paragraph to end of clause	A detailed design procedure covering any arrangement of bolt rows about the beam tension flange, with each bolt row containing two bolts, is given in section 2.8 of *Joints in steel Construction*, SCI Publication No. 207/95, 1997. Guidance on its use in New Zealand, including its use in connections which are subject to inelastic demand, is given on pages 29-32 of the HERA *Steel Design and Construction Bulletin*, Issue No. 56, 2000.

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