Proceedings of Symposium

Fixing, bonding and joining technology in steel buildings & façade 2007

5 January 2007

Edited by

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Ir Professor S.L. Chan

香港
鋼結構學會
Hong Kong Institute of
Steel Construction

The Hong Kong Institute of Steel Construction was established in November 2000. It has been registered as a non-profit making organization with its board members from universities, consultants, contractors and regulatory agents in Hong Kong. The HKISC serves the steel construction industry in Hong Kong and the region and carries the following specific missions:

- Channeling of technology transfer between academic and industry for improved quality in design, analysis and construction;
- Organizing seminars for local and overseas experts for dissemination of their latest technological know-how;
- Organizing international conferences for sharing of expertise between local and overseas researchers and engineers;
- Steering university researches to be more practical and useful for practitioners;
- Disseminating new technology worldwide among members ; and
- Developing and fostering friendship among members.

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DEPT. OF CIVIL AND STRUCTURAL ENGINEERING
香港理工大學
土木及結構工程系

香港
鋼結構學會
Hong Kong Institute of
Steel Construction

Symposium

Fixing , bonding and joining technology in steel buildings & façade 2007

5th January 2007, Friday

Date:	5th January 2007, Friday
Venue:	Harbour Plaza Hotel, Hong Kong No. 20, Tak Fung Street, Hung Hom, Kowloon
Time:	8:45 am (registration) from 9:00 am to 5:00 pm

Time	Program
8:45 am	Registration
9:00 am	Welcoming Speech Mr. H.F. Ren, President, Hong Kong Façade Association
9:10 am	Opening remark Ir Dr C.M. Koon, Chief Structural Engineer, Buildings Department
Lecture 1 9:20 am	***Quality assurance of anchored, bolted and welded jointing materials*** Ir C.K. Cheung, Hong Kong Accreditation Service
Lecture 2 10:00 am	***Glazing solutions with laminated glass beyond the pvb limit: the use of a structural interlayer*** Mr P. S. Davies, DuPont Australia
10:40 am	**Tea Break**
Lecture 3 11:10 am	***Bolted connections in steel structures to the code of practice for structural uses of steel, Hong Kong 2005*** Ir Professor S.L. Chan, The Hong Kong Polytechnic University
Lecture 4 11:50 am	***Protective Structural Sealant Glazing – a new window fixing & bonding technology for protecting building tenant and public safety*** Mr J. Ma, Dow Corning Asia
12:30 pm	**Lunch**
Lecture 5 2:00 pm	***Design and Fabrication of Welded Connections*** Ir Dr W.T. Chan, Buildings Department
Lecture 6 2:40 pm	***Composite joints with steel beams and precast hollowcore slabs*** Dr D. Lam, University of Leeds
3:20 pm	**Tea Break**
Lecture 7 3:50 pm	***Anchor Design for Dynamic Loads - earthquakes and shock*** Dr J. Kunz, Hilti (Hong Kong) Ltd
4:30 pm	Discussion & Closing remark
5:00 pm	**End of Seminar**

CONTENTS

Foreword

Connections are essential structural elements in nearly all steel and metal structures and they include linking two or more structural materials and elements together such as the conventional beam-to-column connections for steel and composite structures, anchor fixing between steel brackets and concrete structures and bonding between two pieces of glass panels. Quality of connections affects greatly the safety and cost of a structure. In many cases, the proper choice of fabrication method and type of connections accelerate the speed of construction which is an important consideration in many construction projects.

This seminar is the first of this kind in Hong Kong jointly organized by The Hong Kong Institute of Steel Construction and the Hong Kong Facade Association with an objective of bringing the latest development on the topic for sharing of knowledge among engineers and builders. We are also honored to have the eminent speakers to share with us the latest development in the field. We further hope that the seminar will draw the attention of architects, engineers, builders and surveyors to the field of connections and facade and steel structures which are becoming more important in contemporary construction.

Mr. H.F. Ren, Mr. Johnny Choi and Prof. S.L. Chan

QUALITY ASSURANCE OF ANCHORED, BOLTED AND WELDED JOINTING MATERIALS

Ir C K Cheung

Hong Kong Accreditation Service

創新科技署

Innovation and Technology Commission

Quality Assurance of Anchored, Bolted and Welded Jointing Materials

Ir C K Cheung
Hong Kong Accreditation Service

Types of connections

- Spider joint between steel frame & safety glass
- Bolted joint between structural sections
- Anchored joint between concrete & steel frame
- Welded joint between structural sections
- Welded joint between rebars
- Coupler joint between rebars

Spider joint between steel frame & safety glass

CODE OF PRACTICE
FOR THE STRUCTURAL USE OF STEEL

BD old requirement

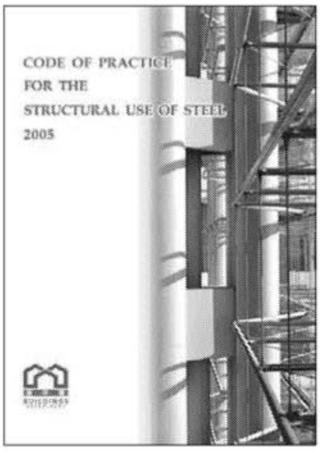
APPENDIX A: LOADING TEST

The methods of testing structures of unconventional design referred to in 5.1.3 of this standard should be as follows:

1. Acceptance tests

The structure or structural member under consideration should be loaded with the dead load for as long a time as possible before testing and the tests should be conducted as follows:

(1) *Stiffness test.* In this test the structure or member should be subjected, in addition to the dead load, to a test load equal to 1.5 times the imposed and wind load, and this loading should be maintained for 24 hours. The maximum deflection attained during this test should not be excessive. If after removal of the test load, the member or structure does not show a recovery of at least 80 per cent of the maximum strain or deflection shown during the 24 hours under load, the test should be repeated. The structure should be considered to have sufficient stiffness, provided that the recovery after this second test is not less than 90 per cent of the maximum increase in strain or deflection shown during the second test. The deflection of a beam under its design loading should comply with 5.6



Proof and strength tests

General

Proof and strength tests are where the structure or component is tested to a particular level of load. A proof test may confirm that the structure performs adequately; a strength test may confirm that it can sustain a particular design load and can be used to accept similar items (see clause 16.3.6). A structure to be strength tested should first undergo a proof test and it is recommended that a failure test should follow the strength test, if appropriate.

Although a proof test is a non-destructive test, there may be permanent local distortions. The effect of these on future use of the structure should be considered before testing. Any departure from linear behaviour during the proof test should be noted and reasons for such behaviour should be explored. A strength test is likely to create significant residual deflection.

The loading steps for both tests are similar. To detect possible creep, the test load should be maintained at as constant value as possible to allow repeated measurements. The loads and deflections should be measured at regular intervals. The intervals should be at least 5 minutes. The loading should be maintained at the proof test load until there is no significant increase in deflection during at least three intervals after the attainment of the test load.

Test loads
The test load for a proof test should be taken as equal to the sum of:

1.0 x (actual dead load present during the test);

and one of the following as appropriate:

a) 1.25 x (imposed load) plus 1.15 x (remainder of dead load);
b) 1.15 x (remainder of dead load) plus 1.2 x (wind load);
c) 1.2 x (wind uplift) minus 1.0 x (remainder of dead load);
d) 1.15 x (remainder of dead load) plus 1.0 x (imposed load and wind load).

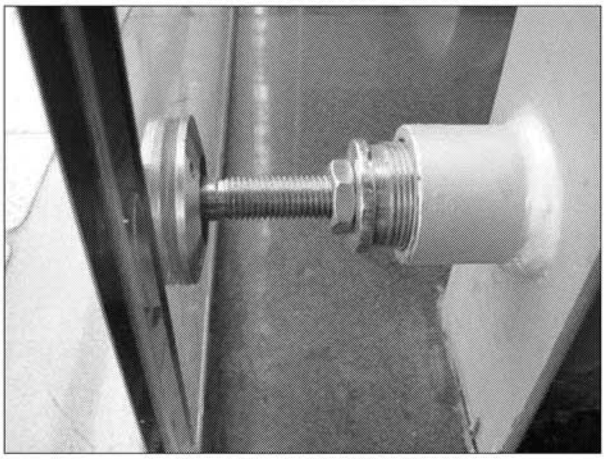
The test load for a strength test is the factored design load (from section 4) multiplied by a relative strength coefficient (see Annex B).

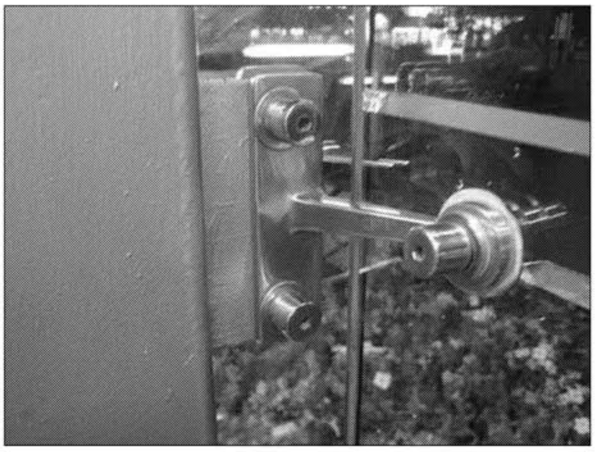
Test criteria
The criteria for a successful proof test are:

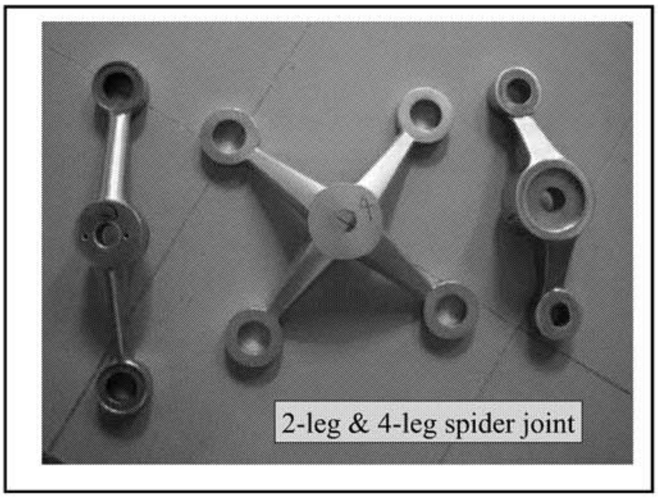
(a) Substantially linear behaviour under the proof test load;
(b) No creep under the proof test load for a period of at least 15 minutes; and
(c) On removal of the test load a residual deflection not exceeding 20% of the maximum deflection recorded during the test.

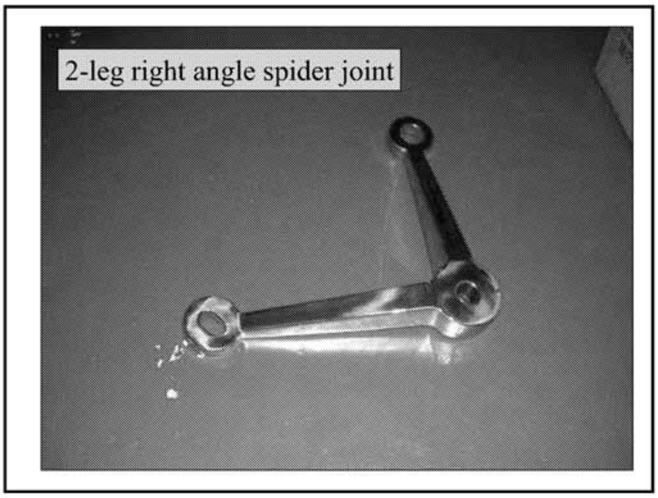
If the proof test is not successful it may be repeated once only. For this repeated test the deflection criteria is reduced to 10% of the maximum recorded during the test.

For a successful strength test the residual deflection on removal of the test load shall not exceed 80% of the maximum deflection recorded during the test and there is no buckling or rupture of any part of the structure.

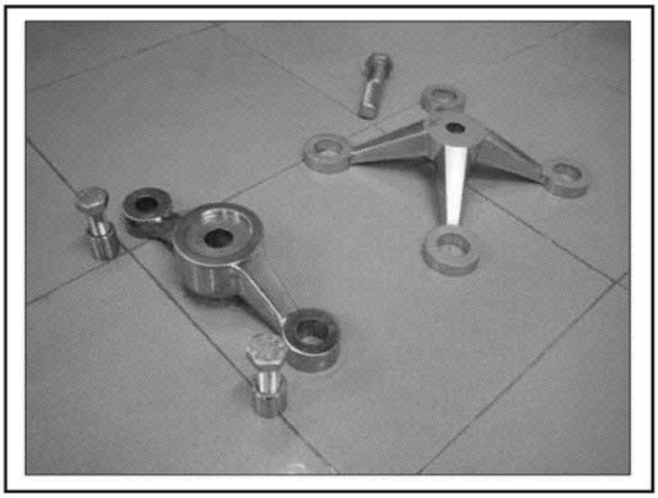


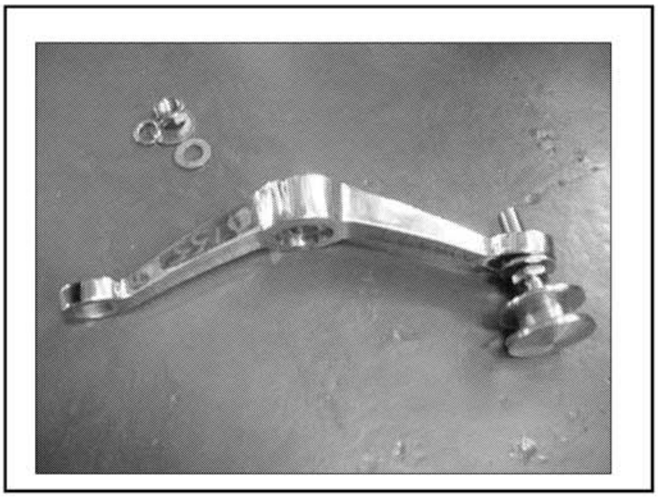


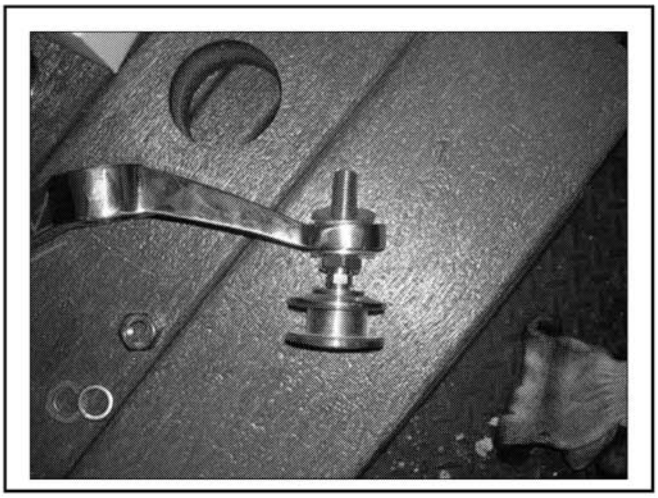

2-leg & 4-leg spider joint


2-leg right angle spider joint

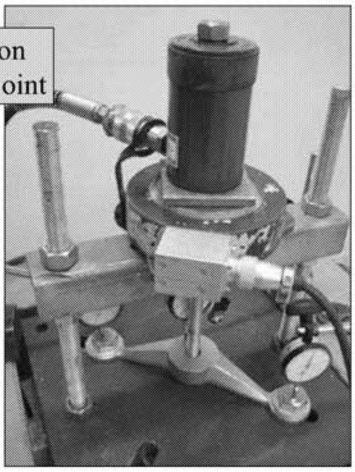

Stainless steel bolts



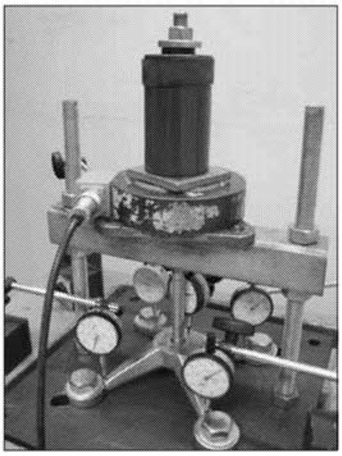


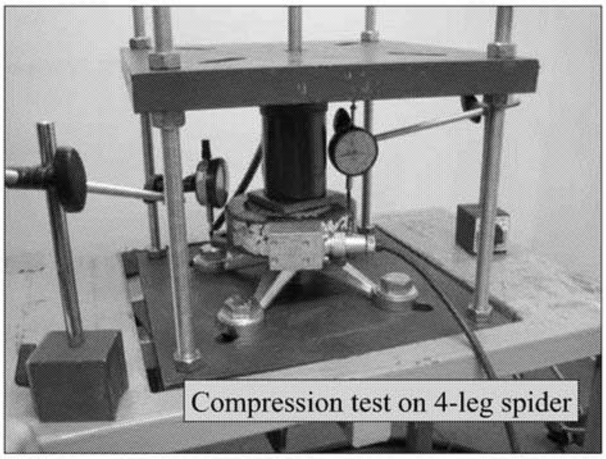


Tension test on
2-leg spider joint



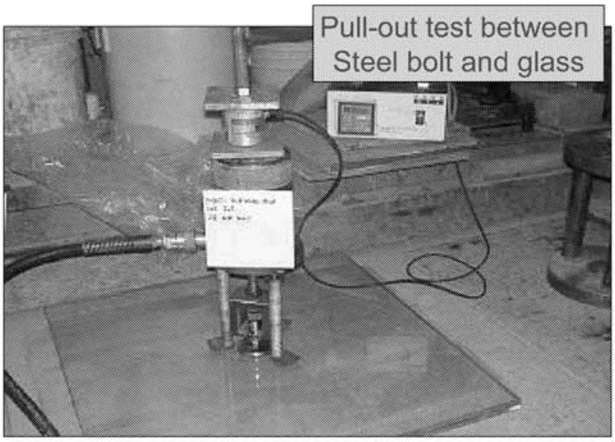
Tension test on
4-leg spider joint





Compression test on 4-leg spider

Pull-out test between
Steel bolt and glass



After testing : failure of glass

Bolted joint
between structural sections


Bolted joints

CODE OF PRACTICE FOR THE STRUCTURAL USE OF STEEL 2005

Normal bolts

Bolts, nuts and washers shall comply with the requirements of the acceptable standards and references given in Annex A1.3.

Bolts with an ultimate tensile strength exceeding 1000 N/mm^2 should not be used unless test results demonstrate their acceptability in a particular design application.

High strength friction grip or preloaded bolts

High strength friction grip bolts, nuts and washers shall comply with the requirements of the reference standards given in Annex A1.3.

Requirements for the design of high strength friction grip bolted connections including tightening procedures are given in clause 9.3.

ABSTRACT OF ESSENTIAL REQUIREMENTS FOR BOLTS

Abstract of essential requirements for bolts:

(a) In a matched assembly of a nut and bolt, the nut must be sufficiently strong enough such that the bolt shank fails in tension prior to the nut or bolt threads stripping.

(b) When bolts and nuts are galvanized, it is usual that the manufacturer will tap the nut threads oversize in order to fit the galvanized bolt threads. Therefore the nut is required to be stronger than for the case when it is not galvanized in order to comply with (a). Typically the manufacturer should supply a higher grade of nut, e.g. a grade 10 nut for a grade 8.8 bolt.

(c) Bolts should only be used in the range of strengths given in Table D4 below unless test results demonstrate their acceptability in a particular design application.

(d) Friction grip bolts may be tightened using the torque control method, part-turn method, or direct tension to BS 7644 or other acceptable standard and the manufacturer's recommendations. Torque spanners and other devices shall be re-calibrated in accordance with BS 4604 or other acceptable standard.

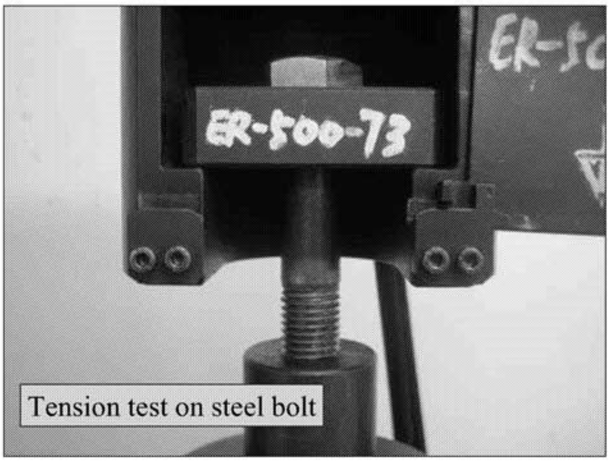
BUILDINGS DEPARTMENT

Table D3 - Essential performance requirements for bolts

Performance requirement	Specified by
Minimum tensile strength	Tensile testing
Minimum yield strength	Tensile testing
Elongation	Tensile testing
Hardness	Brinell Hardness Testing

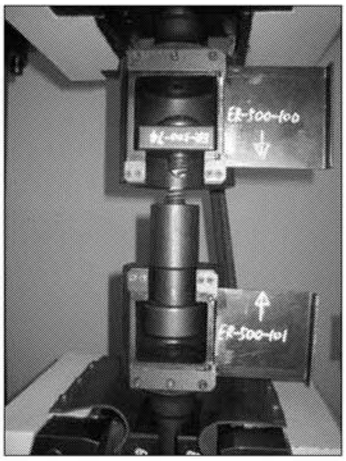
Table D4 - Various normally used bolt strengths

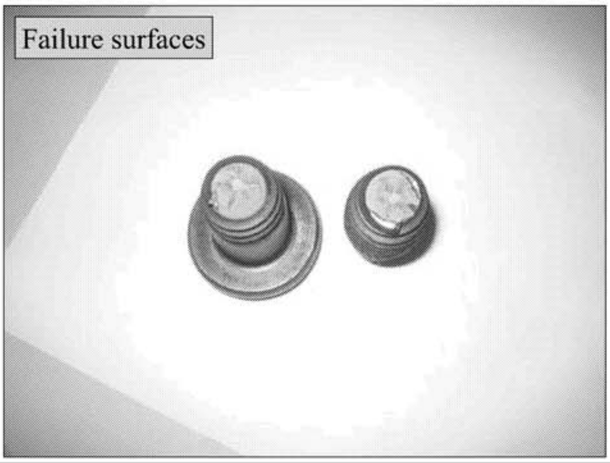
Bolt source and grade	Design shear stress p_s (N/mm^2)	Design bearing stress p_b (N/mm^2)	Design tensile stress p_t (N/mm^2)	Ultimate tensile strength p_u (N/mm^2)
ISO 4.6	160	460	240	400
ISO 8.8	375	1000	560	800
ISO 10.9	400	1300	700	1000
General grade HSFG ≤ 24	400	1000	590	***840***
General grade HSFG ≥ 27	350	900	515	***735***
High strength HSFG	400	1300	700	***1000***
ASTM A307	165	***460***	310	425
ASTM A325	330	***900***	620	855
ASTM A490	415	***1300***	780	1070



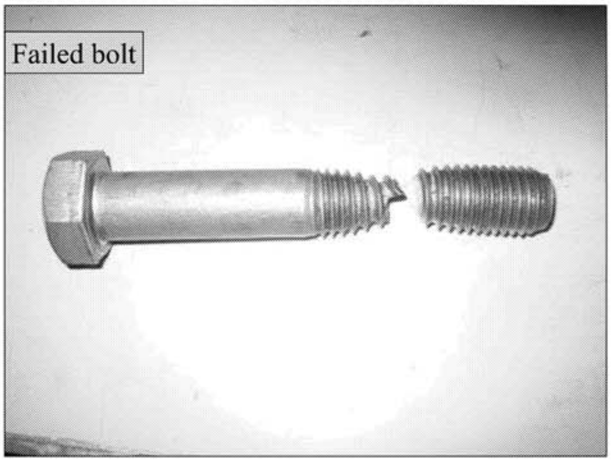
Tension test on steel bolt

Tension failure of the screw

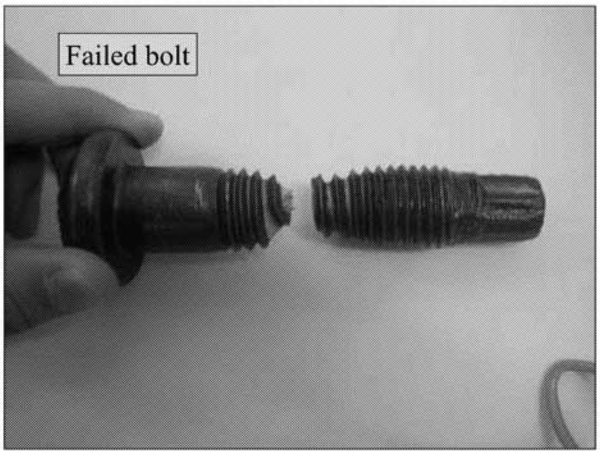




Failure surfaces

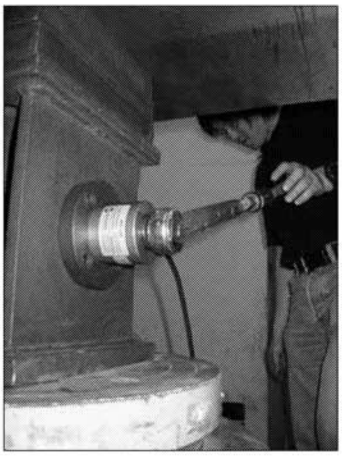


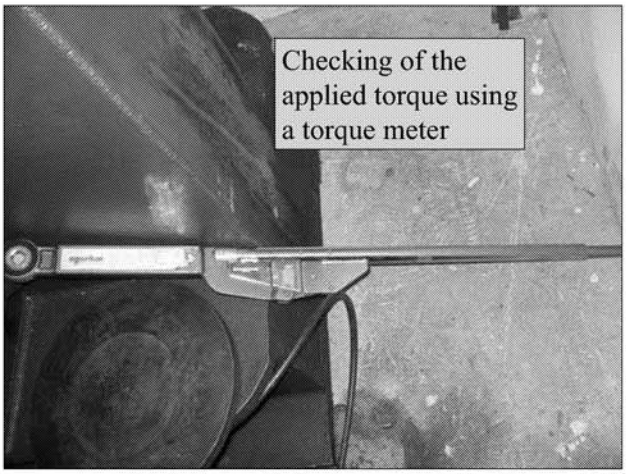
Failed bolt



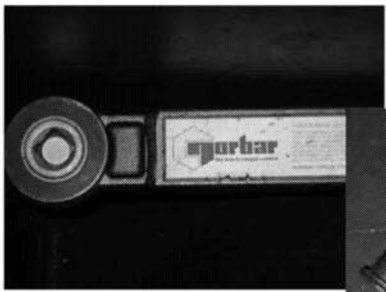
Failed bolt

Checking of the applied torque





Checking of the applied torque using a torque meter



Torque meter

Anchored joint
between concrete & steel frame

PNAP 59 : Cladding

Buildings Department	Practice Note for Authorized Persons and Registered Structural Engineers	59

Cladding

Cladding means a facing or architectural decoration additional to the external walls of building : e.g. aluminium or metal cladding, polished granite slabs, limestone cladding, marble facing and the like. It should comply with the performance requirements stipulated in the Building (Construction) Regulation 39 in respect of material type, fixings, strength and durability.

PNAP 59 : Cladding : Test on anchors

Tests on Anchors

4. On-site strength tests should be carried out on a representative number of each type and size of drilled-in anchors for those parts of cladding above 6 meters from street level. Such tests are necessary to verify the performance and workmanship of the anchors installed and should be carried out under the direction of the registered structural engineer or authorized person.

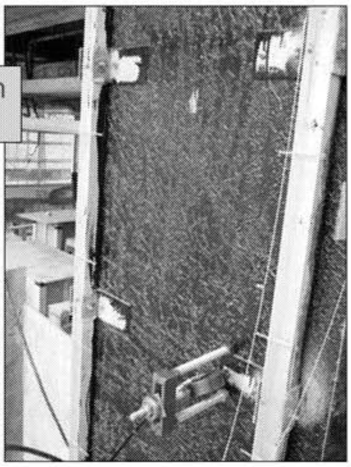
5. Each representative anchor should be tested by means of either :-

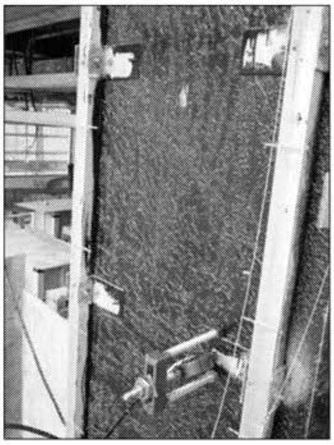
(a) pull-out test; or

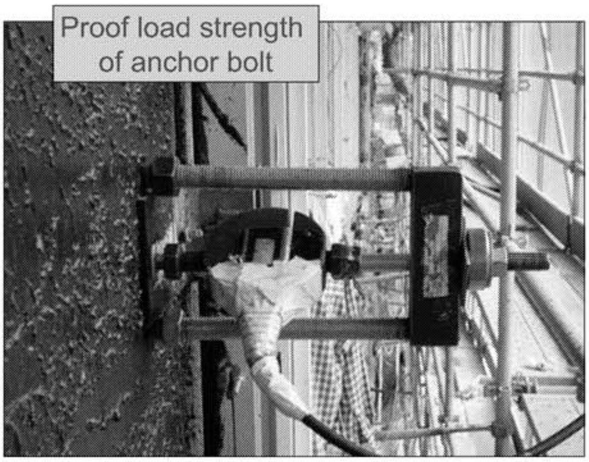
(b) equivalent tightening torque test,

to demonstrate that its pull-out capacity is not less than 1.5 times the recommended tensile load as specified by the anchor manufacturer. The tested anchor should be considered satisfactory if it does not show any signs of separation, plastic deformation or deleterious effect during the test.

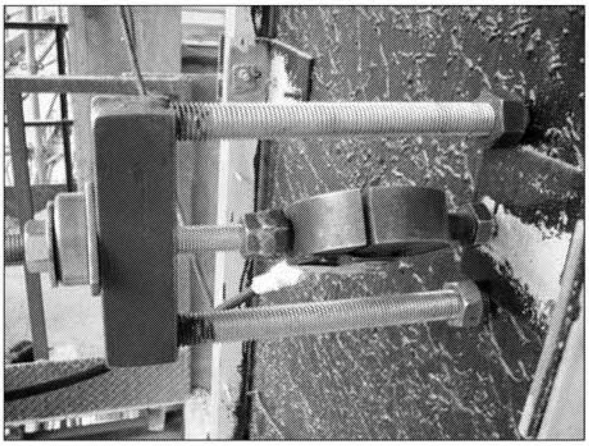
Proof load strength
of anchor bolt





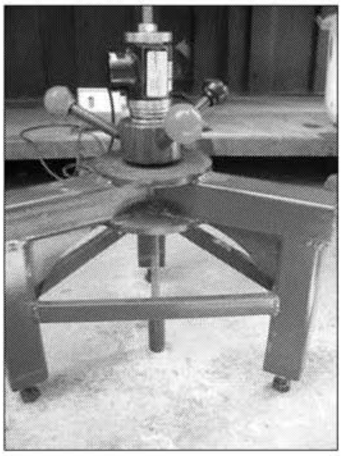


Proof load strength of anchor bolt



Pullout strength of anchor bolt

BS 5080-1:1993



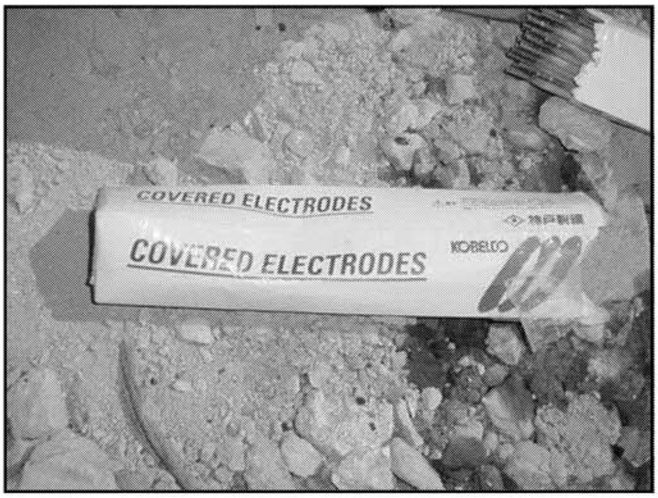
Welded joint between structural sections

Types of electrodes

BUILDINGS DEPARTMENT

WELDING CONSUMABLES

All welding consumables shall conform to the requirements of the reference standards given in Annex A1.4. For steel with design strength not exceeding 460 N/mm^2 the specified yield strength, ultimate tensile strength, elongation at failure and Charpy energy value of the welding consumables shall be equal to or better than the corresponding values specified for the grade of steel being welded. The most onerous grade shall govern if dissimilar grades are welded together. For high and ultra-high strength steels, the welding material may, if necessary to produce a suitable joint, be of a lower strength; the elongation to failure and Charpy impact value should still match those of the parent material. In that case, the design strength of the weld must be based on the weld material.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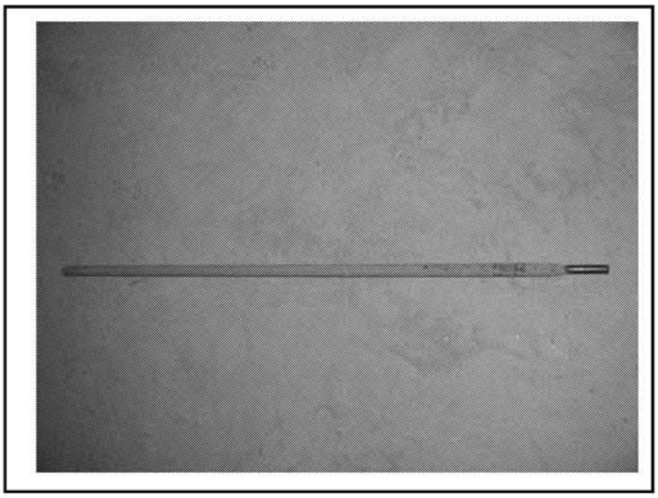
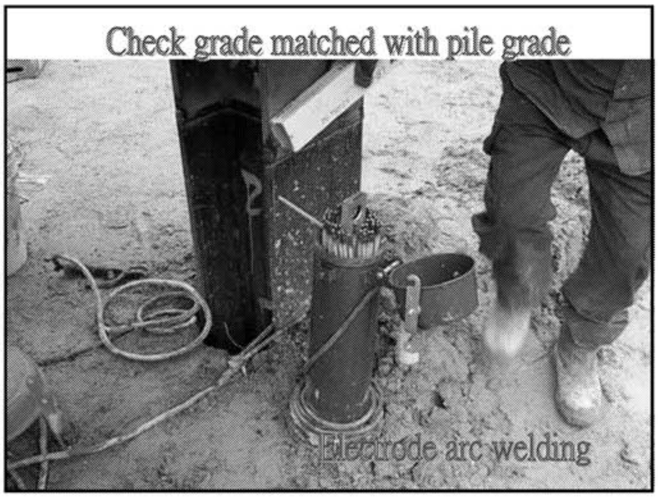
COVERED ELECTRODES
KOBELCO




LC-600
5.0φ×400mm
Approved by
KR.DNV.BV.NK.GL
朝鮮線材熔接棒
CHO SUN
WELDING
ELECTRODES
NET 5 kgs
4 BOX


LC-600
CHO SUN
WELDING
ELECTRODES

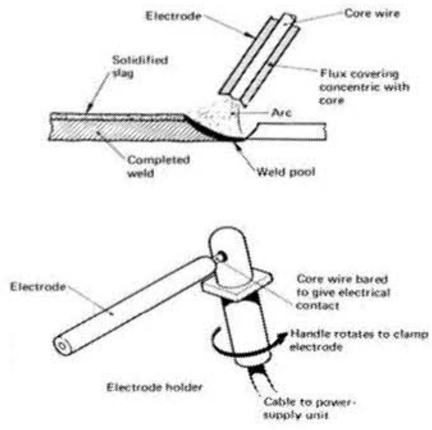

Check grade matched with pile grade
Electrode arc welding

Kobelco(Japan) – KOBE Steel LB-52

- 3.2mm diameter & 350mm long
- Standard- AWS A5.1 E7016
- BS EN : 499 E42 3 B & JIS D5016
- Type of covering: low hydrogen
- Use: 490MPa Class high tensile & mild steel
- Current: Flat 90-130A, Vertical & overhead 80-120A
- Polarity of electrode AC or DC+

Cho Sun Electrode, Korea LC-600

- 5mm diameter & 400mm long
- Approved by KR, DNV, BV, NK, GL
- Standards: ASME SFA-5.5,
- AWS A5.5 E9016-G,
- KS D7006 D5816,
- JIS Z3212
- Current: 150-240A, AC or DC+

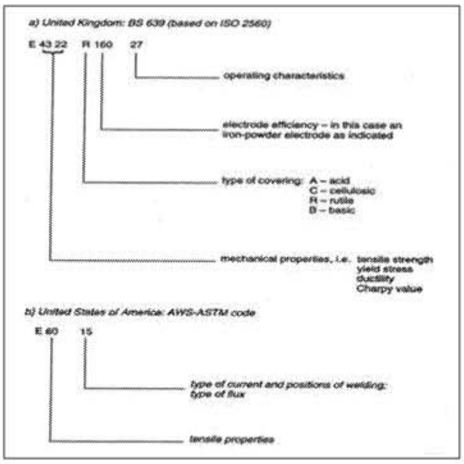


Schematic diagram of MMA welding

MMA electrode constituents and their functions

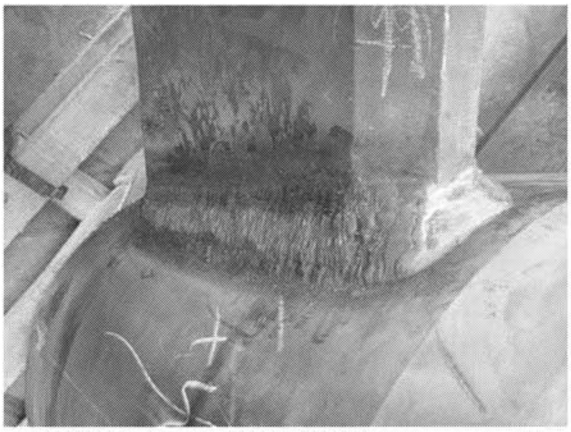
Constituent	Primary function	Secondary function
Iron oxide	Slag former	Arc stabiliser
Titanium oxide	Slag former	Arc stabiliser
Magnesium oxide	Fluxing agent	———
Calcium fluoride	Slag former	Fluxing agent
Potassium silicate	Arc stabiliser	Binder
Other silicates	Slag formers and binders	Fluxing agents
Calcium carbonates	Gas former	Arc stabiliser
Other carbonates	Gas former	———
Cellulose	Gas former	———
Ferro-manganese	Alloying	Deoxidiser
Ferro-chrome	Alloying	———
Ferro-silicon	Deoxidiser	———

Note Binders are used to give the flux covering mechanical strength and to help it adhere to the core wire. Fluxing agents are used to adjust surface tension and wetting characteristics.



Steel Weld Test

Special
Pile shoe



Testing required for welder approval EN 287-1:1997

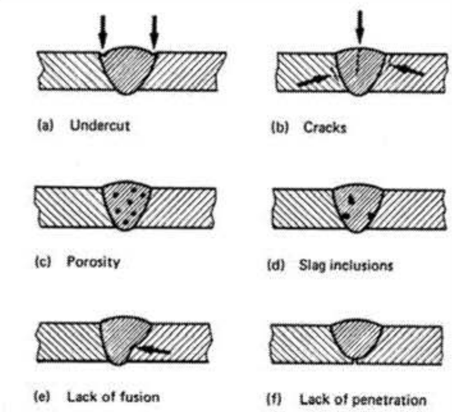
- Visual - butt weld plate & pipe, fillet
- Radiography – butt weld plate & pipe
- Fracture - butt weld plate & pipe
- Bend – butt weld plate & pipe
- Macro – not mandatory
- Magnetic/penetrant
- Ultrasonic – replace radiographic if thickness of plate > 8 mm

Testing of test pieces for welding procedure approval EN 288-3:1997

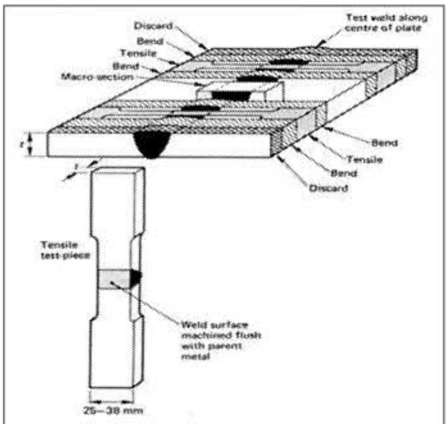
- Butt weld – visual, radiographic or ultrasonic, transverse tensile & bend, impact, hardness & macro examination
- T-butt – visual, RT or UT, hardness, macro examination
- Fillet – visual macro examination & hardness

Destructive tests on welds

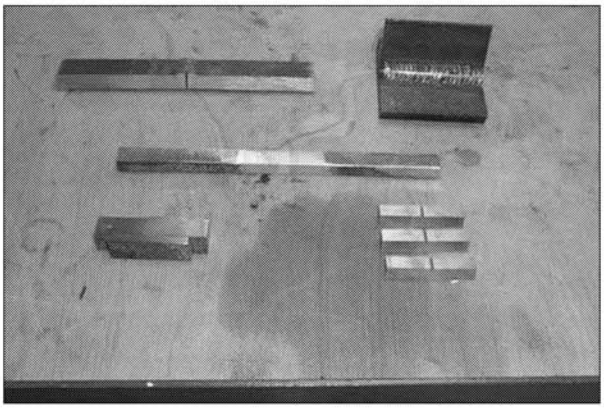
- Transverse tensile test
- Transverse bend
- Macro examination
- Fracture
- Impact
- Hardness

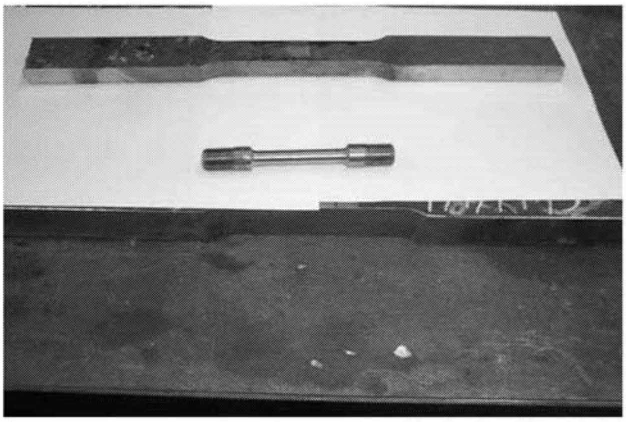


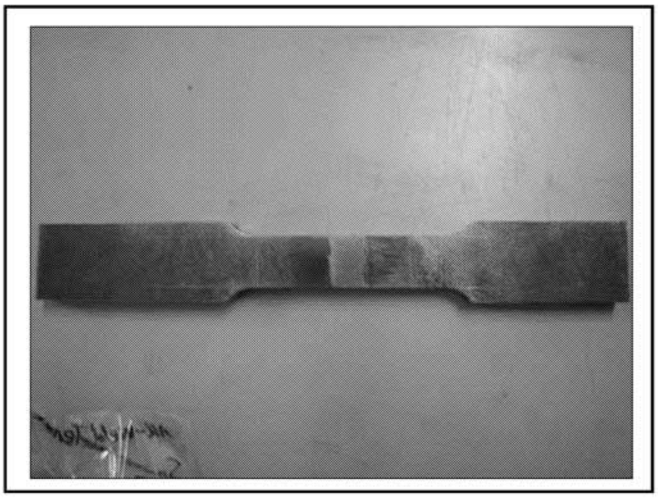
Cross-sections of welds containing typical defects

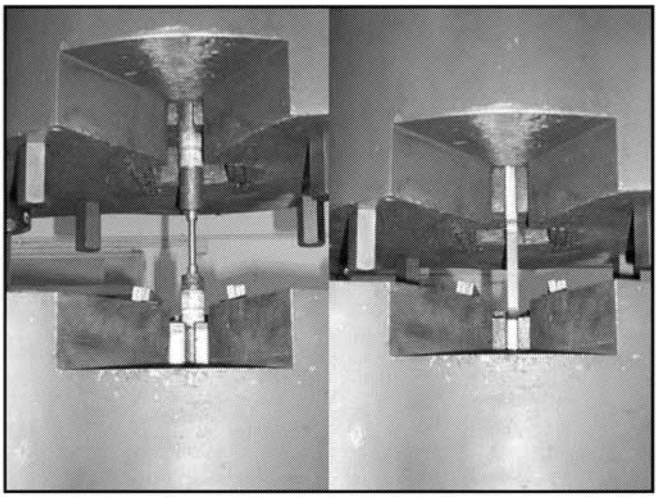


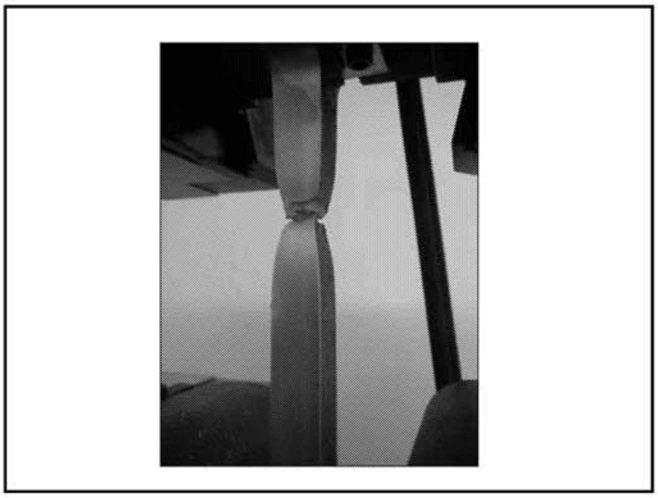
Method of cutting test-pieces from procedure-approval plate

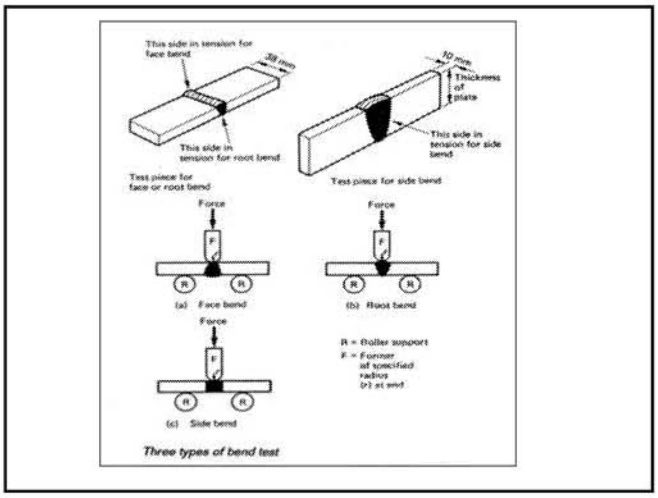




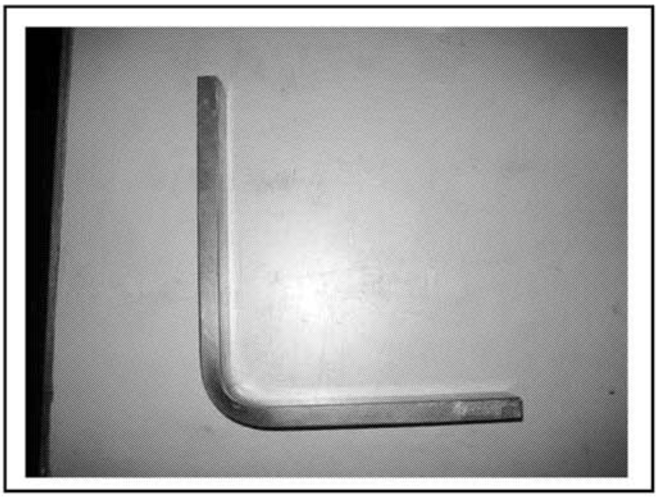


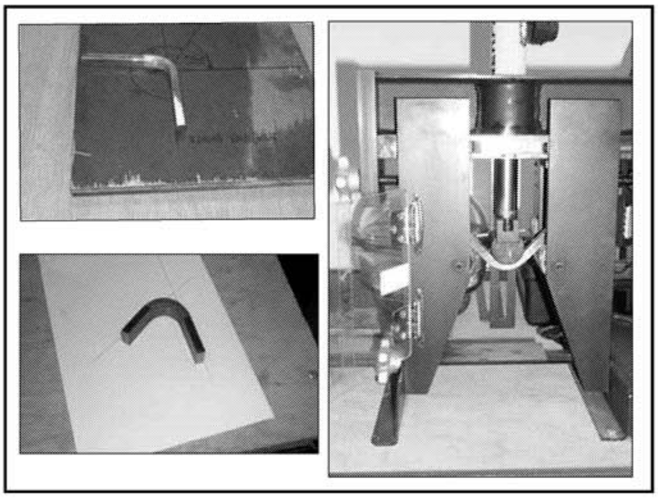


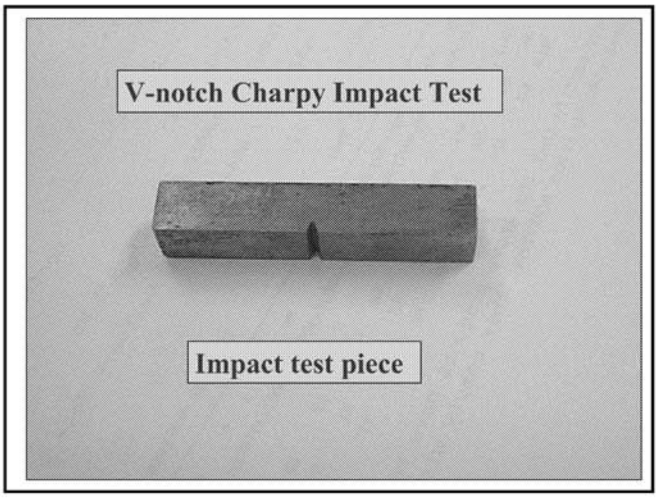


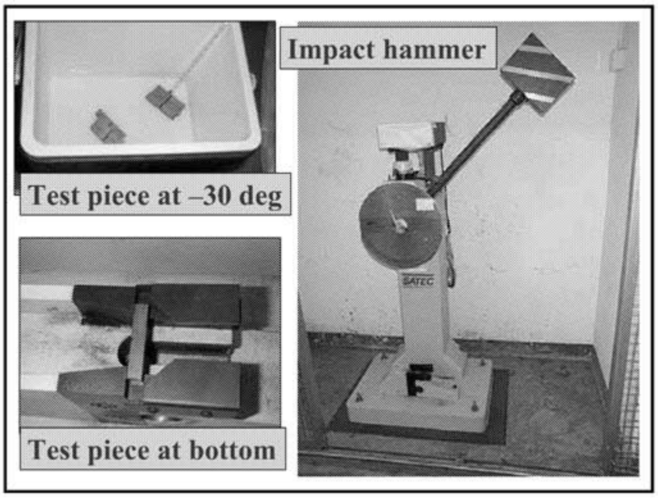


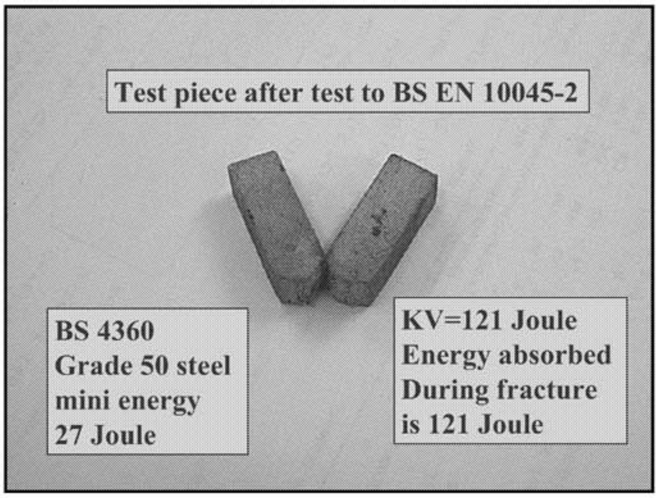
This side in tension for face bend
38 mm
This side in tension for root bend
Test piece for face or root bend
10 mm
Thickness of plate
This side in tension for side bend
Test piece for side bend
Force
F
R
(a) Face bend
(b) Root bend
(c) Side bend
R = Roller support
F = Former of specified radius (r) at end
Three types of bend test

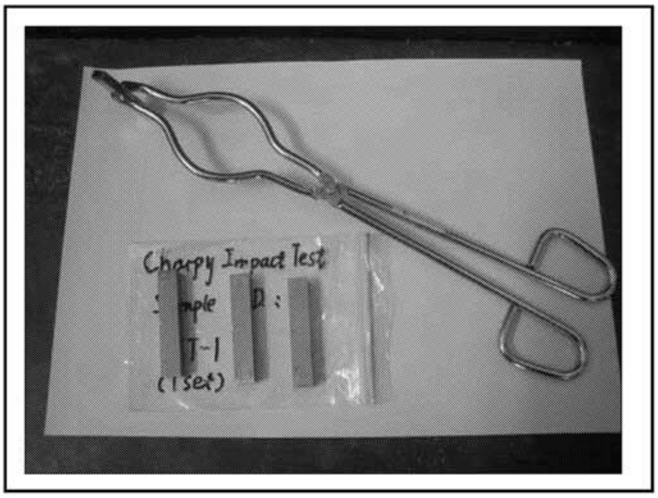


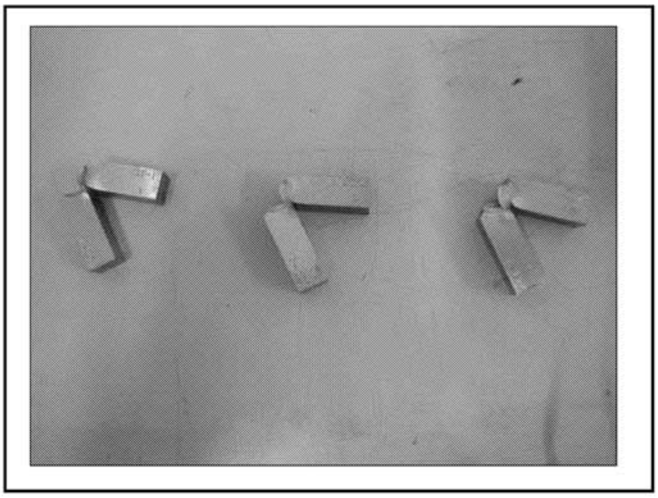



V-notch Charpy Impact Test
Impact test piece

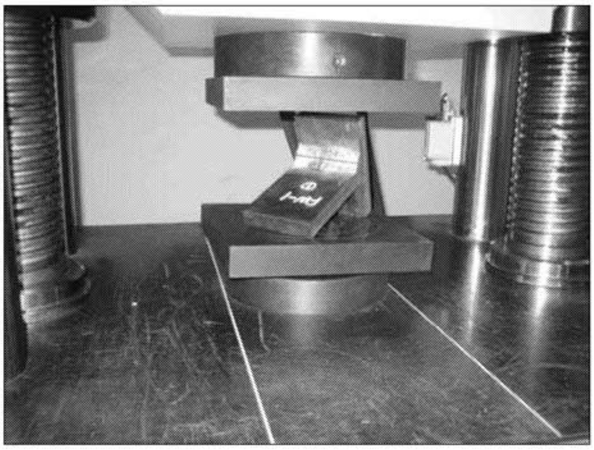

Impact hammer
Test piece at –30 deg
Test piece at bottom
SATEC


Test piece after test to BS EN 10045-2
BS 4360
Grade 50 steel
mini energy
27 Joule
KV=121 Joule
Energy absorbed
During fracture
is 121 Joule


Charpy Impact Test
T-1
(1set)




FW-1

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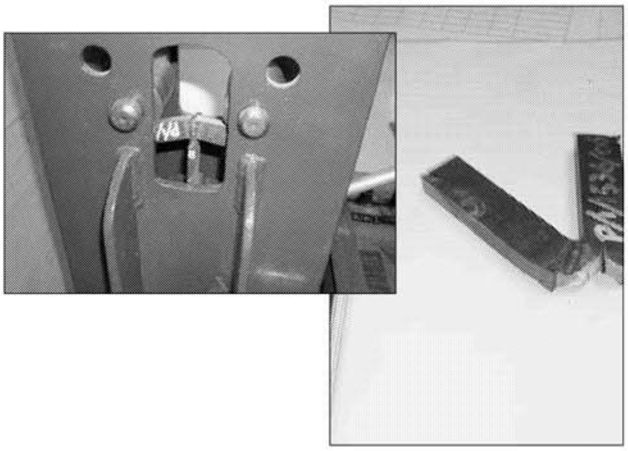
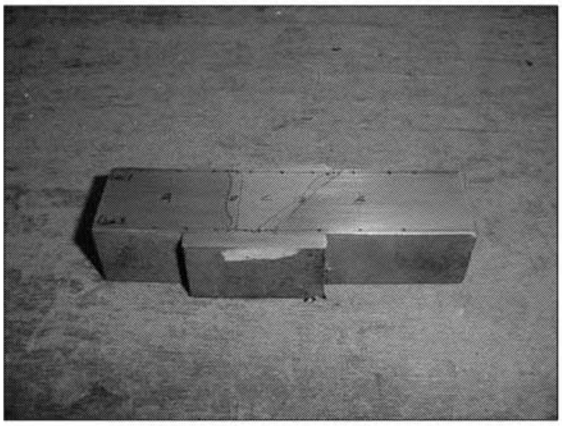
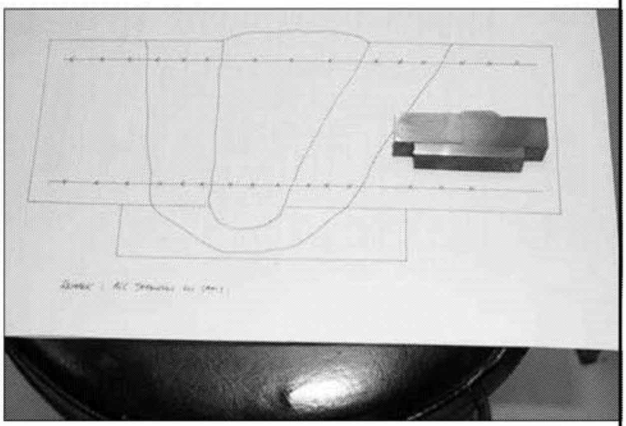
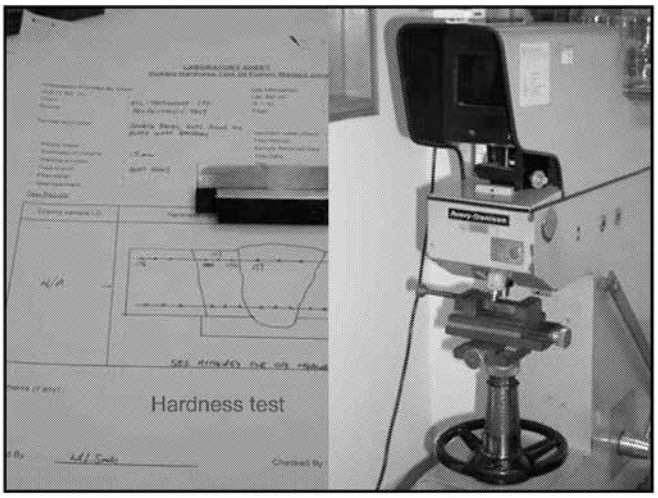
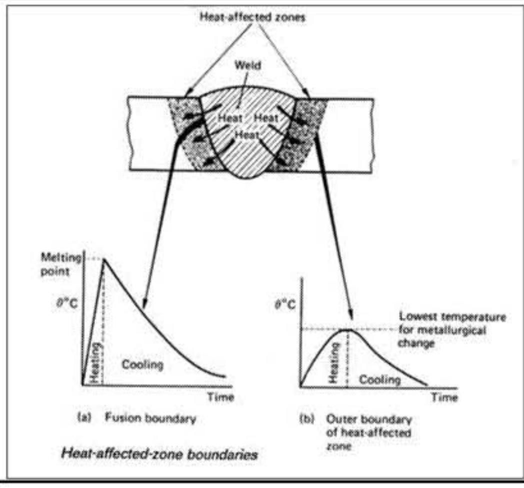
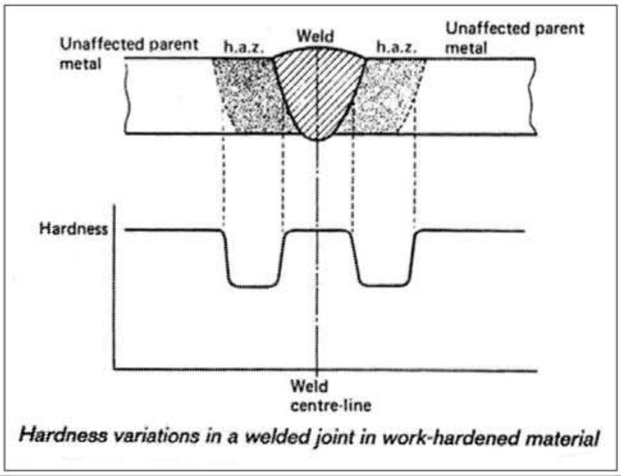












Non-Destructive Tests on Welds

Non-destructive Test on Welds
(HOKLAS Supplementary Criteria No.15)

- Ultrasonic
- Radiography (X-ray & Gamma-ray)
- Magnetic Particle
- Dye Penetrant
- Visual examination

Approved HOKLAS Signatory Level III ASNT or PCN

Approved HOKLAS Operator Level II ASNT or PCN

American Society of Nondestructive Test

British Institute of Nondestructive Test

HOKLAS Supplementary Criteria No. 15

Construction Materials Test Category – Accreditation of Non-destructive Tests for Welding of Steel and Metal

1 **Introduction**

1.1 This Supplementary Criteria is an amplification and interpretation of the requirements of HOKLAS 002 and 003 for the accreditation of non-destructive tests (NDT) for welding of metal and steel under the Construction Materials Test Category. The non-destructive test covers the following methods:

Ultrasonic testing (UT)

Magnetic Particle testing (MT)

Liquid Penetrant testing (PT)

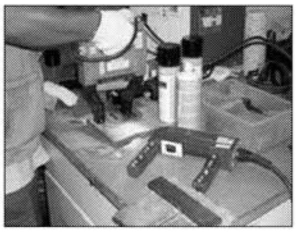
Radiographic testing (RT)

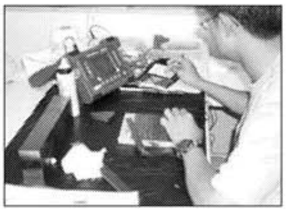
Visual Examination (VE)

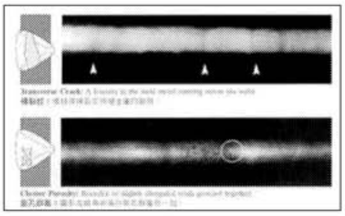
Issued in Jan 2000

1.2 Laboratories for NDT shall operate under a quality system complying with HOKLAS 003. This document gives specific requirements with respect to personnel, equipment, calibration, test methods, handling of testing items, measurement uncertainty and proficiency testing. For aspects not mentioned in this document, reference should be made to HOKLAS 002 and 003. Laboratories shall also note that complying with this document may not necessarily meet the requirements of all test standards. Individual test standards may have specific requirements which must be met when conducting the tests.

THE AMERICAN SOCIETY FOR NONDESTRUCTIVE TESTING

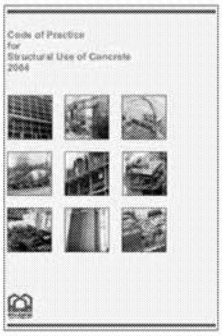






Welded joint
Between reinforcements bars

Welding of Reinforcement Steel Bars



BS 7123

Weldability

Reinforcement may be considered weldable provided the types of steel have the required welding properties given in the acceptable standards. Welding to be inspected and approved by a competent person.

Where the weldability is unknown, tests should be carried out.

8.7 LAPS AND MECHANICAL COUPLERS

8.7.1 General

Forces are transmitted from one bar to another by:

- lapping of bars, with or without bends or hooks;
- welding; or
- mechanical devices assuring load transfer in tension-compression or in compression only.

In joints where imposed loading is predominantly cyclical bars should not be joined by welding.

10.4.6 Welding

10.4.6.1 General

Generally, welding should be carried out under controlled conditions in a factory or workshop. Welding on site should be avoided if possible.

Welding must only be carried out on reinforcing steel that has the required welding properties.

Welded connections must be made and checked by suitably qualified persons. The competence of the welders should be demonstrated prior to, and periodically during, welding operations.

All welding should be carried out in accordance with the appropriate acceptable standards and the recommendations of the reinforcement manufacturer.

Reinforcing bars should not generally be welded at or near bends.

BD Reinforced Concrete Code

10.4.6.2 Use of welding

Welding may be used for the following purposes:

- fixing in position, for example, by welding between intersecting bars, or between bars and other steel members. Metal-arc welding or electric resistance welding may be used on suitable steels; or
- structural welds involving transfer of load between reinforcement or between bars and other steel members. Butt welds may be carried out by flash butt welding or metal-arc welding. For lapped joints, metal-arc welding or electric resistance welding may be used.

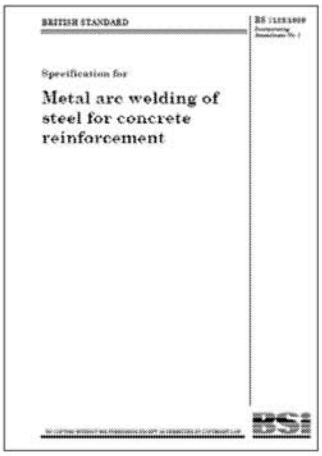
10.4.6.3 Types of welding

Types of welding permitted include:

- metal-arc welding;
- flash butt welding;
- electric resistance welding; or
- other methods.

Other methods of welding may be used subject to the approval of the client and reinforcement manufacturer.

BS 7123


BRITISH STANDARD

Specification for

Metal arc welding of steel for concrete reinforcement

Table 1 — Minimum preheating temperatures for butt joints and cruciform joints: hydrogen controlled consumables[a]

	Minimum preheat temperature		
Nominal bar size mm	25	> 25 to 40	> 40
Carbon equivalent %	°C	°C	°C
0.42 or less	0	0	50
> 0.42 to 0.51	50	75	100

[a] Those depositing weld metal containing not more than 15 mL of hydrogen/100 g deposited metal when assessed by the relevant standard.

BS 7123

Table 2 — Minimum preheating temperatures for butt joints and cruciform joints: non-hydrogen controlled consumables

	Minimum preheat temperature		
Nominal bar size mm	25	> 25 to 40	> 40
Carbon equivalent %	°C	°C	°C
0.42 or less	50	75	100
> 0.42 to 0.51	100	hydrogen controlled consumables to be used	

BS 7123

12.3 Welding procedure approval tests

12.3.1 *General.* The following tests shall be carried out:

a) full section tensile test;

b) macroetch test.

12.3.2 *Welding procedures.* All joints for procedure approval shall be welded in the position that is to be used in practice, that is with the axes of the reinforcing bars horizontal or vertical as shown in Figure 10 and Figure 11.

12.3.3 *Number and type of tests.* The number and type of tests required for procedure approval shall be as given in Table 6 and carried out on test pieces and test specimens shown in Figure 12.

Welding Procedure Approval tests

Table 6 — Tests required for procedure approval

Joint type	Number and type of tests	
	Tensile tests	Macroetch tests
Butt joints	2	1
Splice joints	2	1
Lap joints	2	1
Cruciform joints	2	1
Tack welded lap joints	—	1
Tack welded cruciform joints	—	1

BS 7123

13 Welder approval

13.1 General

The contractor shall satisfy the client of his welder's ability to make sound welds.

13.2 Changes not affecting approval

The following changes between the approved welding procedure and the procedure used for the welder approval tests shall not entail re-approval of the welder:

a) a change in grade or type of reinforcing bar;

b) a change in weld metal composition;

c) a change from a basic to a rutile covered electrode;

d) a change in preheating temperature.

Welder Approval tests

Table 9 — Tests for welder approval

Test piece	Number and type of tests	
	Tensile test	Macroetch test
Butt joint	1	1
Lap joint	1	1
Splice joint	1	1
Cruciform joint	1	1
Tack welded lap joint	—	1
Tack welded cruciform joint	—	1

BS 7123

HKIAS Supplementary Criteria No. 3

Construction Product Inspection - Welding Inspection

HKAS

1 Introduction

1.1 This document serves to amplify and interpret the requirements of HKIAS 003 for the accreditation of welding inspection. The welding inspection covered includes inspection/approval of welding procedures, inspection/approval of welders testing, inspection of welding processes and welds quality. The following sections set out specific technical criteria for selected clauses of HKIAS 003. For areas not covered in this document, the principles stipulated in HKIAS 003 shall apply.

1.2 Accreditation is granted for specific types of inspection (e.g. pre-welding activities, during and after welding processes, etc.) for specific types of welds and using specific inspection methods or standards. Inspection bodies may apply for accreditation for one or more inspection methods or standards and one or more welding types. Accreditation for generic inspection methods covering a broad range of different welds for the same types of welding inspection process may also be granted. The welding inspection activities covered by the generic inspection method or standard will be detailed in the scope of accreditation.

Welding Inspection Personnel

- HKIAS Approved Welding Inspector
- Certified Welding Inspector of the American Welding Society, or American Welding Society Gateway to the World of Welding
- Welding Inspector under CSWIP CSWIP
- HKIAS Approved Signatory for Welding Inspection
- Senior Certified Welding Inspector of AWS
- Senior Welding Inspector of CSWIP

BS 7123

Table 7 — Acceptance levels (visual and macro examination)

Imperfection type	Permitted maximum	
	Procedure approval and Welder approval (visual and macro-examination)	Production welding (visual examination)
a) Cracks	Not permitted	Not permitted
b) Lack of root fusion[a] Lack of side fusion Lack of inter-run fusion	Not permitted	Not applicable
c) Lack of root penetration[a]	[a]Not permitted	Not applicable
d) Undercut	Depth not to exceed 1 mm	Depth not to exceed 1 mm
e) Excess weld metal	Weld metal to blend smoothly with the parent metal	Weld metal to blend smoothly with the parent metal
f) Overlap	Not permitted	Not permitted
[a] Applies to butt joints only.		

Table 8 — Acceptance levels (radiographic examination) for butt welds

Imperfection type		Permitted maximum
Cavities	a) Isolated pores (or individual pores in a group)	Diameter not greater than 0.125 of bar size and 3 mm maximum
	Distributed porosity	3 % of projected area[a].
Solid inclusions	Slag inclusions	maximum length 10 mm maximum width 2 mm
[a] Projected area is the length of the weld affected by the porosity times the maximum width of the weld.		

BS 7123

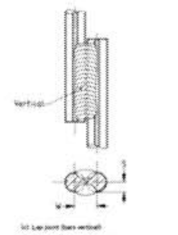


Figure 4 — Lap joints

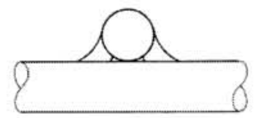


Figure 6 — Cruciform joint

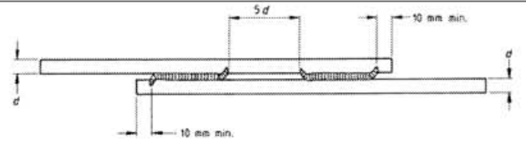
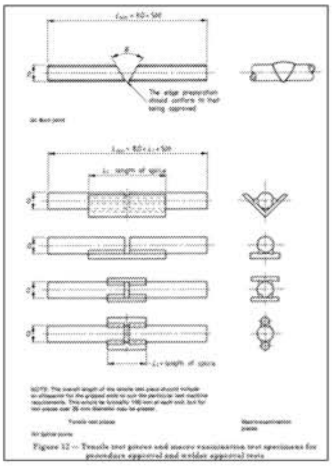
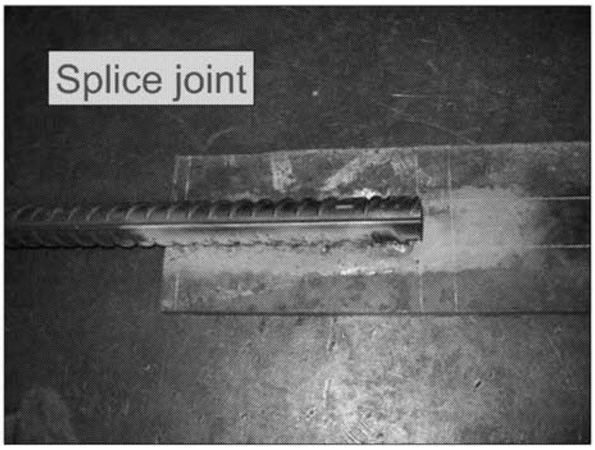
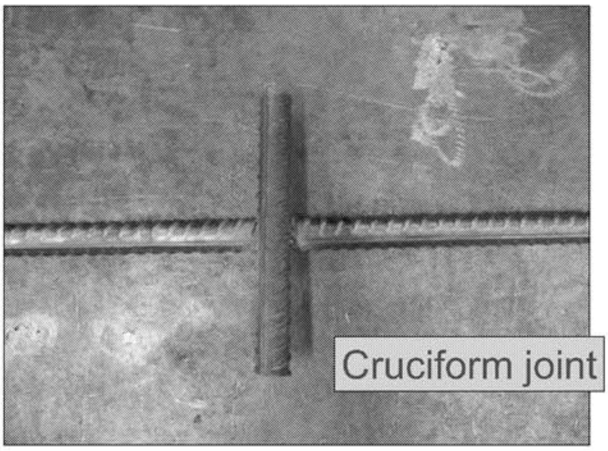


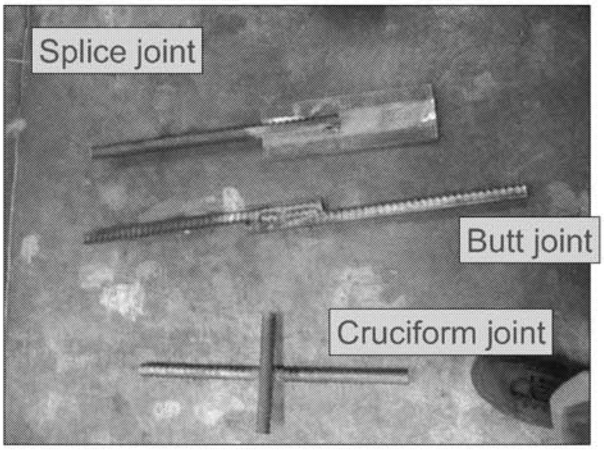
Figure 5 — Start and finish of a lap joint

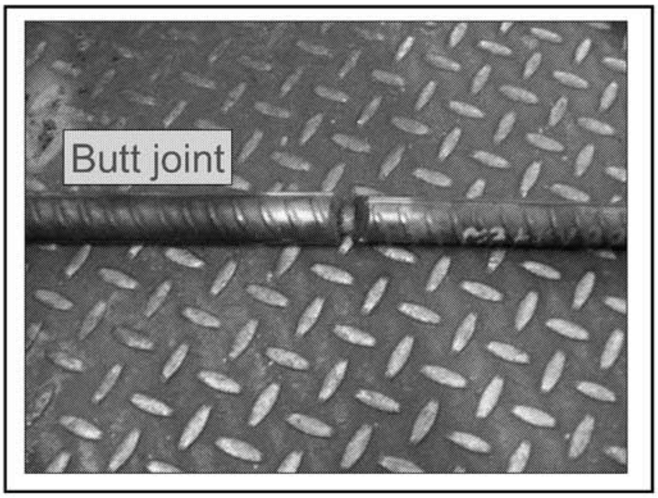
BS 7123

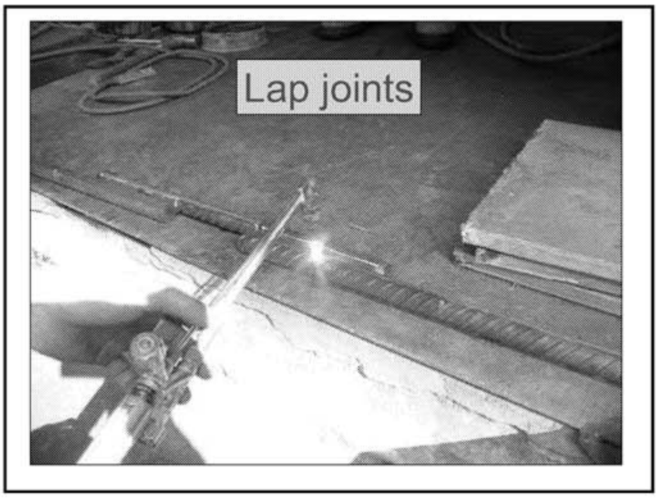


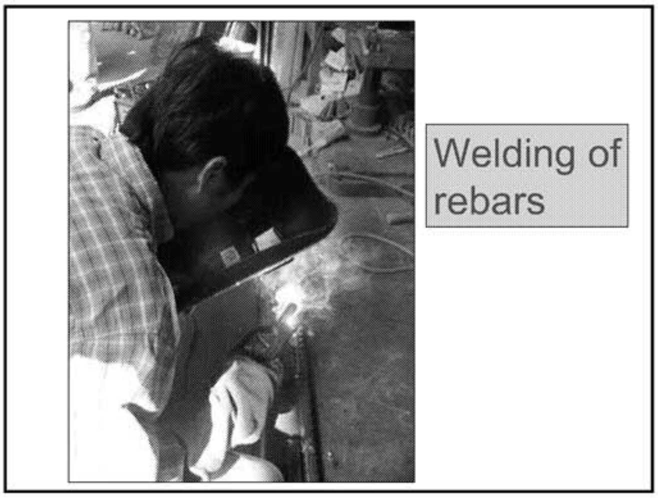


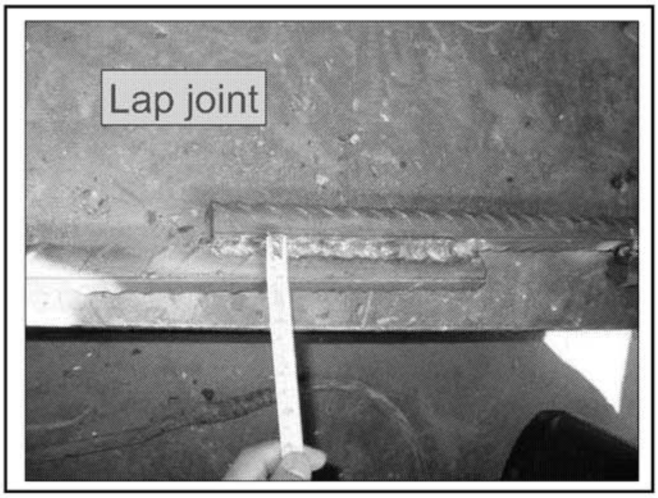


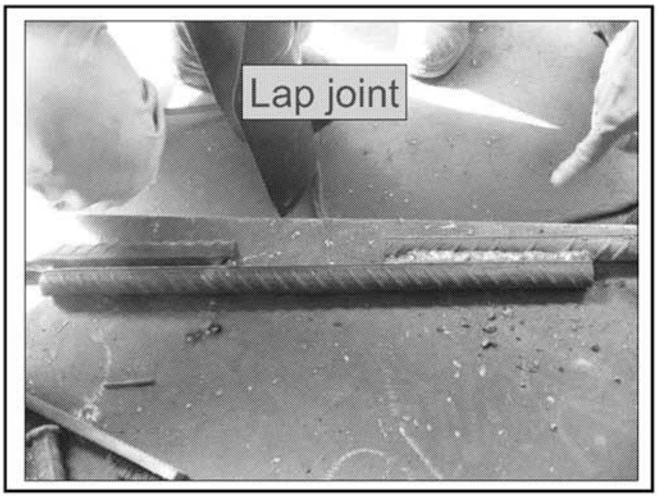


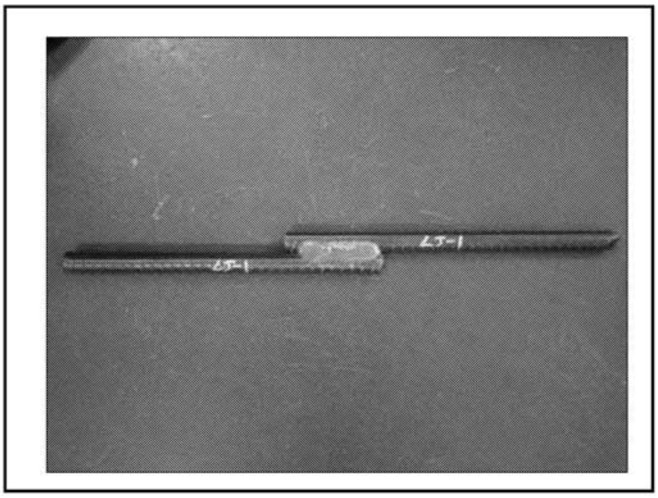

Butt joint

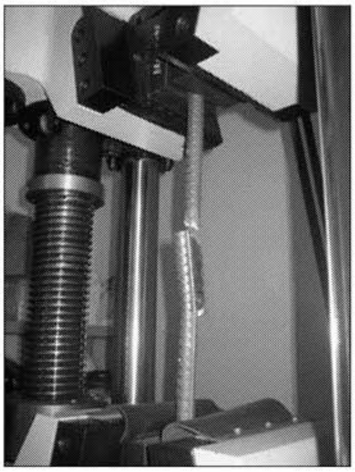

Lap joints


Welding of rebars

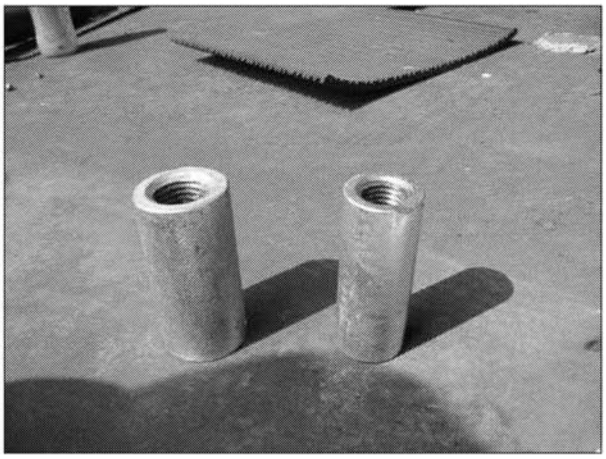

Lap joint

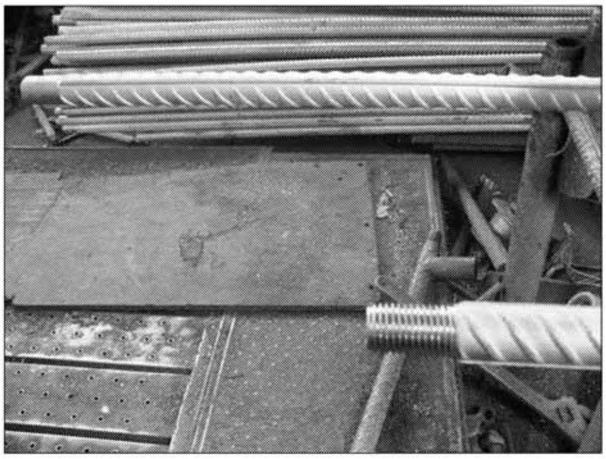

Lap joint

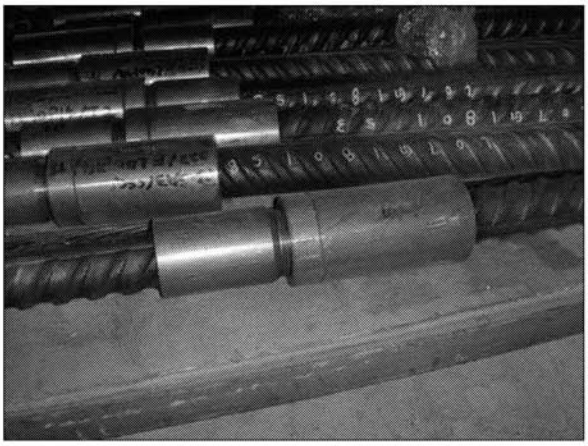


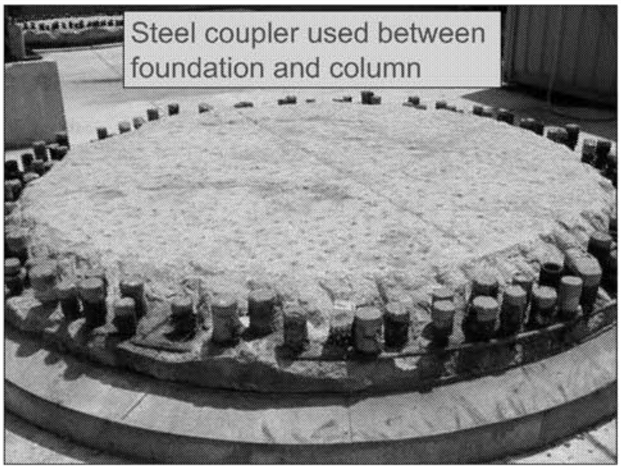


Coupler joint
between reinforcement bars









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BUILDINGS DEPARTMENT

3.2.8 Mechanical couplers

3.2.8.1 Bars in compression

The load may be transferred between butt jointed bars by means of end bearing where sawn square cut ends are held in contact by means of a suitable sleeve or other coupler. The concrete cover to the sleeve should not be less than that specified for normal reinforcement.

3.2.8.2 Bars in tension

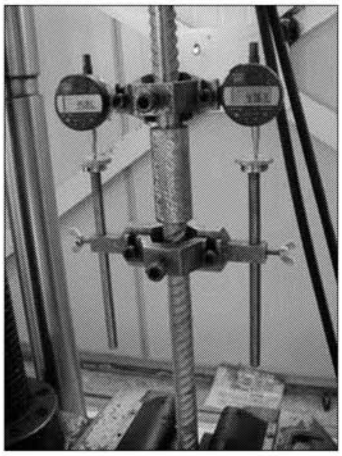
The only acceptable full strength butt joint between bars in tension is formed using a mechanical coupler satisfying the following criteria:

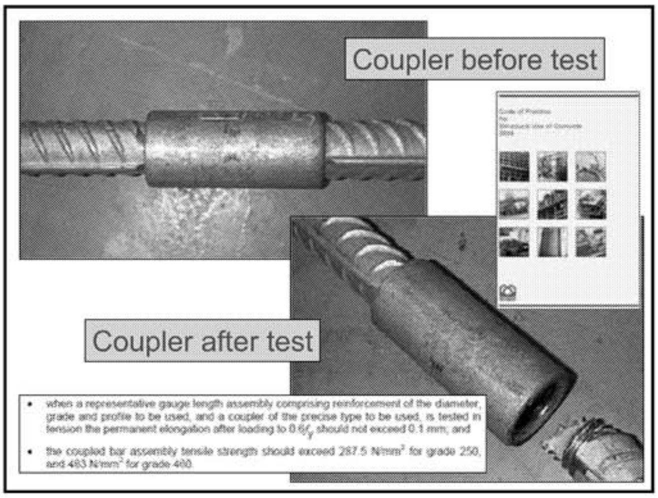
- when a representative gauge length assembly comprising reinforcement of the diameter, grade and profile to be used, and a coupler of the precise type to be used, is tested in tension the permanent elongation after loading to $0.6f_y$ should not exceed 0.1 mm, and
- the coupled bar assembly tensile strength should exceed 287.5 N/mm^2 for grade 250, and 483 N/mm^2 for grade 460.

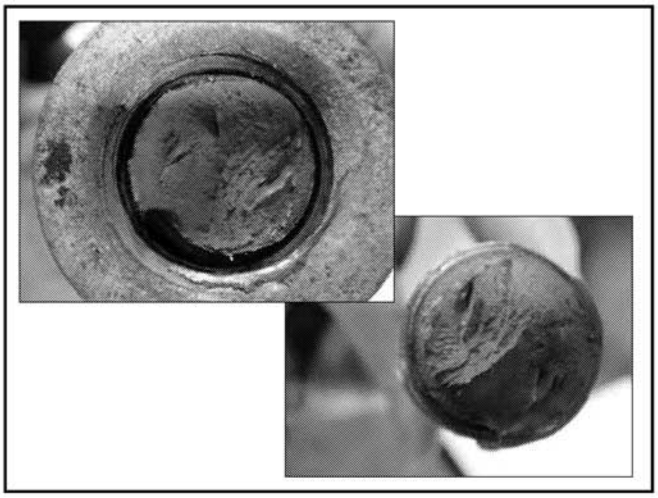
Tensile test to BS8110

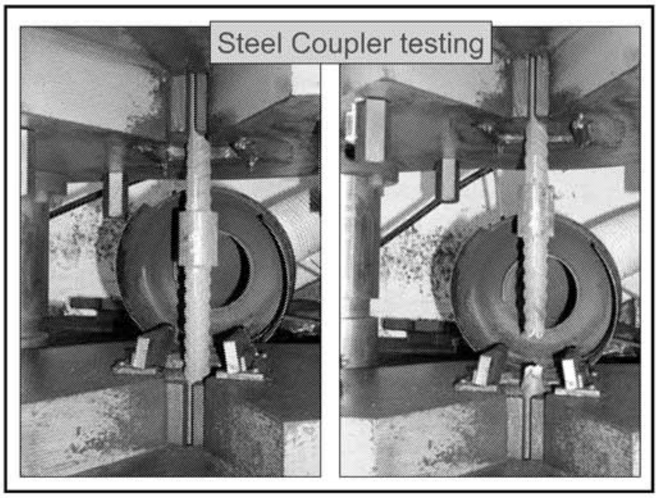
Load to 0.6 fy unload to zero

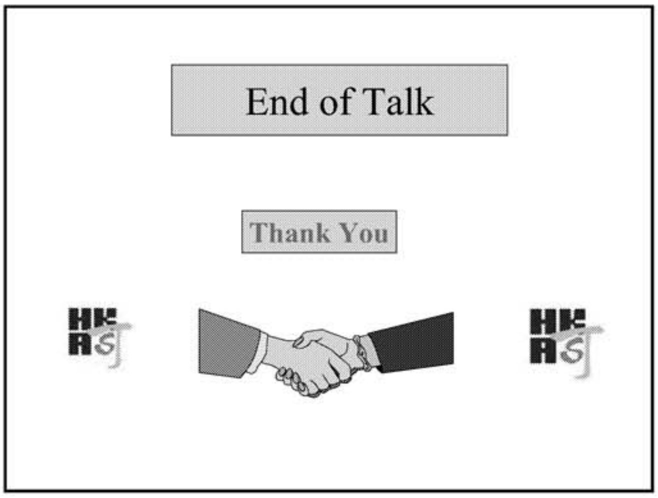
Permanent deformation < 0.1 mm




Coupler before test
Coupler after test
• when a representative gauge length assembly comprising reinforcement of the diameter, grade and profile to be used, and a coupler of the precise type to be used, is tested in tension the permanent elongation after loading to $0.6f_y$ should not exceed 0.1 mm; and
• the coupled bar assembly tensile strength should exceed 287.5 N/mm^2 for grade 250, and 483 N/mm^2 for grade 460.




Steel Coupler testing


End of Talk
Thank You

GLAZING SOLUTIONS WITH LAMINATED GLASS BEYOND THE PVB LIMIT: THE USE OF A STRUCTURAL INTERLAYER

Stephen J. Bennison, Phillip S. Davies, William Gao, Frank Feng Zhu & Tammy Amos

Glass Laminating Solutions
E.I. DuPont de Nemours & Co. Inc
Wilmington, DE 19880-0356, USA

ABSTRACT

In this contribution we look at some of the more challenging applications for laminated glass in architecture and show how solutions may be provided using a structural, non-PVB, interlayer. Enhancements in mechanical performance, such as strength, creep, post-glass breakage coupled with enhancements in aesthetics, durability and materials compatibility, are provided with a SentryGlas® Plus structural interlayer that extends the performance of laminated glass well beyond the established PVB limits. Examples are given of projects where such a structural interlayer provides key enabling technology to the design solution.

1. INTRODUCTION

Architectural laminated glass is dominated by the use of PVB interlayers, such as DuPont Butacite®. This ascendancy can be traced to the long (> 60 years), successful history of PVB use in the automotive industry for laminated safety glass windshields. Although many requirements for automotive laminates and architectural laminates are the same, there are notable differences. The main distinction is the need for post-glass breakage compliance, in automotive applications to reduce the likelihood of head trauma in an accident. Also, the demands of performance longevity for architectural applications generally exceed those needed for automotive applications. If we relax the need for post-glass breakage compliance (softness) and scrutinize the performance needs for architectural applications, we quickly realize that polymers with enhanced structural, temperature and durability performance are attractive for architectural applications.

Here we examine the use of one such structural interlayer: DuPont SentryGlas® Plus. This interlayer is based on a different chemistry to PVB and has been developed from a class of DuPont proprietary polymers. SentryGlas® Plus was introduced some seven years ago to meet the needs for high-performance hurricane resistant glazing in the United Sates. In recent years we have seen rapid growth in the use of SentryGlas® Plus in non-hurricane applications, as the structural and durability advantages of this interlayer have been realized.

2. SOME PERFORMANCE LIMITS FOR PVB-BASED LAMINATES

The performance limits for PVB-based laminated glass are generally well known and in some cases defined clearly in national standards. For example, ASTM E1300-04 (Load Resistance of Glass in Building), uses design charts to map the strength of laminates under wind load. The charts show that for short-duration loading up to 50 ºC in *four-side* supports, PVB-laminates show essentially equivalent strength to monoliths for common applications. However, in all cases where support is *less than* four sides, for example in frameless glazing (point support), PVB laminates are always weaker than equivalent monoliths and exhibit greater deflection for a given load. Elevated temperatures and long duration loads further challenge the load transfer of the PVB interlayer resulting on sub-monolithic performance. Invariably, design solutions require the use of thick glass to compensate for the lack of load transfer across the PVB interlayer and are especially noticeable in minimally supported glazing, such as bolted glass or where long load duration requirements at elevated temperatures dictate the design criterion. The compliance and creep characteristics of PVB laminates after breakage require special detail to be placed on glass type selection and attachment methods to take advantage fully of the PVB performance.

A common concern with PVB laminates is the potential for the development of edge-related features. Although not usually a structural concern, clouding of the edge region or systematic delaminations detract from the aesthetics and are particularly objectionable for open-edged, or butt-joined glazing. Edge features with PVB can be avoided with careful laminate processing and the avoidance of certain environmental conditions, such as an exposed edge being immersed in standing water.

Finally, we see more demand for incorporating metallic structures, such as meshes or perforated sheets inside a laminate. Compatibility and processing issues challenge the use of PVB interlayers in such constructions.

3. NEW SOLUTIONS WITH A STRUCTURAL INTERLAYER

Here we present several examples of real structures in which the use of SentryGlas® Plus has provide an elegant design solution that has extended the laminate performance beyond a PVB solution and in many instances, reduced the total solution cost.

4. High Strength Laminates: The Planar™ | SentryGlas® Plus System

A key benefit of the mechanical properties of a SentryGlas® Plus interlayer is the full structural coupling achieved between glass plies in the laminate. These structural properties are maintained to elevated temperatures and long-term load durations. This provides much enhanced strength behavior and reduced laminate deflection versus a conventional PVB laminate. The result of such efficient structural performance is reduced thickness laminates, increased panel sizes, lighter supporting structure and generally lower overall system weight. Often, this reduced weight takes significant cost out of a façade and the use of thinner tempered glass plies in the laminate offers further opportunities for cost savings.

The structural properties of SentryGlas® Plus have been fully utilized by Pilkington engineers in the development of the Planar™ | SentryGlas® Plus System. Frameless laminated glass based on Planar™ | SentryGlas® Plus provides designers with the most structurally efficient glazing systems available, i.e. minimum laminate thickness, larger glass spans and fewer support fittings.

Figure 1 Planar™ | SentryGlas® Plus System in the Yorkdale Mall barrel vault roof (Toronto, Canada).

An example of a Planar™ | SentryGlas® Plus System can be seen in Figure 1, which shows the barrel vault roof of the Yorkdale Shopping Mall in Toronto, Canada. The snow load specification for this roof was 65 pounds per square foot (3.1 kPa). To meet this specification with a PVB interlayer for the desired panel size, an insulated glass unit (tempered pane plus laminated pane) with a total glass thickness of 35 mm would have been needed. With the Planar™ | SentryGlas® Plus System one pane of 10 mm thick glass and one pane of a 17.5 mm laminate, i.e. 26 mm total glass, has been used. This is a weight saving of over 25 % for the units.

A further example of a Planar™ | SentryGlas® Plus System is shown in Figure 2. This is part of a façade for a police station in Hong Kong and was designed with various security requirements. A feature of this job is the use of the Pilkington Integral™ attachment method in which the supporting bolt connects to the inner glass ply of the laminate leaving a smooth external ply of glass.

Figure 2 Planar™ | SentryGlas® Plus System in a Police Station (Hong Kong).

5. POST-GLASS BREAKAGE PERFORMANCE

A further advantage of the structural properties of SentryGlas® Plus is the improved mechanical performance of a laminate after glass breakage. The stiffness advantage over PVB, which is maintained to high temperatures, results in a more robust condition if the laminate is broken accidentally.

An example of glazing with a post-breakage requirement in its design is the 3,000 m^2 glass roof of the global headquarters of energy giant Endesa in Madrid, Figure 3. The Endesa building is seven floors high and the glass atrium roof is positioned some 24 m above ground level. The glass panels were selected for the best aesthetics and strength are 2.7 m x 1.35 m; the glass roof has an overall weight of 135 tonnes.

Figure 3 SentryGlas® Plus laminated glass roof structure of the Endesa Building (Madrid, Spain).

Bellapart Engineering and DuPont conducted a comprehensive range of strength tests on Endesa's laminated roof structure. These included maximum expected load; impact tests (involving a soft body of 50 kg falling 2.5 m onto the glass construction); post-breakage tests and temperature tests - primarily to satisfy fire regulations - where the glass construction, even after glass breakage, was shown to withstand temperatures of up to 80 °C caused by hot smoke for 15 minutes.

6. COMPATIBILITY WITH METALS

An exciting new trend in the façade industry is the use of glass in conjunction with metallic screens. Screens may be incorporated into insulated glass units or embedded directly into laminated glass. Such screens produce intriguing aesthetics and are effective in controlling light transmission and the managing energy performance of the glazing. The premier example of an embedded mesh is in the façade of the Shanghai Oriental Arts center, Figure 4.

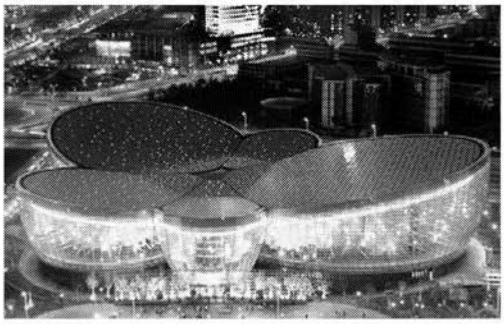


Figure 4 SentryGlas® Plus laminated glass façade in the Shanghai Oriental Arts Center (Shanghai, China).

The glass consists of a perforated sheet steel mesh embedded into the SentryGlas® Plus interlayer, Figure 5. SentryGlas® Plus was chosen for superior structural performance, compatibility with the mesh and processing characteristics that allows the interlayer to readily fill the perforations in the sheet.

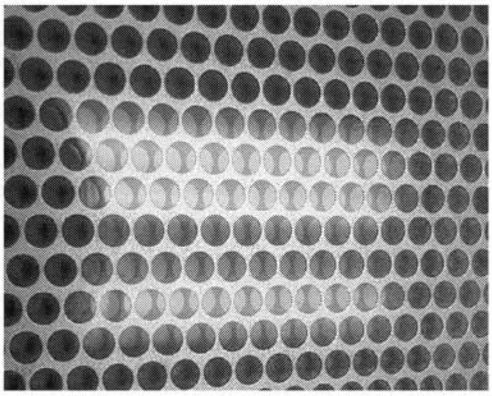


Figure 5 SentryGlas® Plus laminated glass with an embedded perforated metal screen (used in the Shanghai Oriental Arts Center).

7. DURABILITY

Extensive product testing and surveys of actual SentryGlas® Plus-containing laminates in service for over seven years in Florida have shown that this interlayer maintains its mechanical/structural performance and is less prone to the development of edge features typical of many other type of laminated glass. For example measurements of bending stiffness for laminates exposed to varying weathering, up to seven years natural exposure in Florida (temperature excursions between 0 to 45 ºC, and relative humidity up to 90 %) show the bending stiffness is essentially unchanged over time and is direct evidence of the interlayer durability. The interlayer also passes the USA Florida Dade County requirements of 5 years accelerated weathering with less than 10 % change in physical properties.

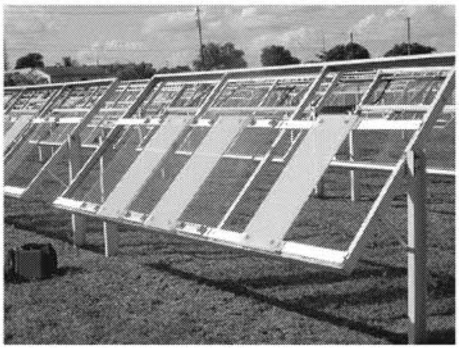


Figure 6 DuPont weathering station showing testing of SentryGlas® Plus laminated glass panels (Florida, USA)

Figure 6 is an image of various laminates undergoing weathering and silicone compatibility testing at one of our weathering stations in Florida. The results of this and many other studies have led DuPont to offer and industry first for an interlayer supplier: a warranty program for laminators covering certain types of edge feature. Approved DuPont laminators and their customers can feel confident in the superior edge performance of laminates made with SentryGlas® Plus versus other interlayer types.

Figure 7 SentryGlas® Plus laminated glass balustrades, Citizen's Bank Park (Philadelphia, USA).

A perfect example of an application that takes full advantage of the strength, durability and edge performance of SentryGlas® Plus laminates is the use in balustrade systems. Figure 7 shows one use of such laminates in cantilevered balustrades at a sports stadium. The need for the most structurally efficient laminate with exposed edges drove the selection of SentryGlas® Plus.

8. AESTHETICS: ULTRA-WHITE LAMINATES WITH "LOW-IRON" GLASS

Many architects now call out the use of low-iron glass for an aesthetically pleasing whiteness and clarity. Most interlayers exhibit a small degree of yellowness that becomes noticeable when used in conjunction with low-iron glass.

Figure 8 SentryGlas® Plus laminated with low-iron glass to produce an ultra-white laminate with exceptional clarity

The polymer system used as the basis for SentryGlas® Plus exhibits ultra-low yellow, YID < 2 (versus PVB, YID = 6 – 12) leading to an exceptionally whiteness and clarity when used to laminate low-iron glass. Figure 8 shows an example of the aesthetic that may be attained.

9. DESIGN INFORMATION

Much of the structural performance benefits of SentryGlas® Plus have yet to be incorporated into national codes and standards. DuPont provides technical information and support for designers interested in using SentryGlas® Plus. Two calculation tools are currently available for general use. The first is a free on-line calculator that may be accessed: www.dupont.com/safetyglass. The on-line calculator allows a user to compare the behavior of laminated glass made with SentryGlas® Plus to PVB for simple cases of beams under various loading/support conditions. The calculator estimates glass stress and laminate deflection for different loads, temperatures and interlayers and shows the conditions under which a structural interlayer provide opportunities for lighter, structurally efficient, cost-effective design solutions. The second commercially available software tool is available at: www.standardsdesign.com and provides estimates of load resistance for glass and laminated glass following ASTM E1300-03. The software also allows interlayer selection again showing optimum designs for a specified loading/support condition. Finally, for design assistance beyond the conventional, DuPont has developed finite element methods that allow deformation analysis for complex loading/support conditions. The finite element methods are based upon accurate models of interlayer mechanical properties, both SentryGlas® Plus and Butacite® (PVB) for a broad range of temperatures and loading histories.

10. CONCLUSIONS

We have shown that the established performance and design limits for PVB-based laminated glass may be surpassed with the use of a structural interlayer, such as SentryGlas® Plus. Not only does this interlayer expand the opportunities for architects to design facades with new levels of performance but also provides opportunities for improved cost-effective structures.

SOME NEW CONSIDERATIONS ON DESIGN OF BOLTED CONNECTIONS TO CODE OF PRACTICE FOR STRUCTURAL USES OF STEEL 2005, HONG KONG

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1. INTRODUCTION

The Code of practice for structural uses of steel 2005 was introduced in summer of 2005 and it will be implemented for full use end of this year. About the part of connections, a few new elements are introduced which include the consideration of prying action and block shear failure. This paper gives a brief summary on the theory behind these two checks.

The most commonly used grades of bolts are grade 4.6, 8.8 and 10.9 bolts. Other grades of bolts are 4.8, 5.6, 5.8, 6.8 and 12.9. Bolt strength outside the range of 400 and 1000 should not be used unless test confirms the applicability. Not only strength of bolts, but also the ductility, hardness and quality control on uniformity should be considered when selecting bolts for a project. Both ordinary clearance bolts and High Strength Friction Grip (HSFG) bolts are widely used to date.

Grade 4.6 bolts made of low carbon high strength steel are used normally for medium and light duty connections such as purlins or sheeting. Holding down bolts also commonly use grade 4.6 bolts because of preferred ductility not only on bolts, but also on base plate of which the design strength is not allowed to be greater than $270N/mm^2$.

Grade 8.8 bolts or higher grade bolts made of high strength alloy should be used for heavy duty connections. HSFG (High Strength Friction Grip) bolts should be used in the load reversal condition and in case when the controlled deflection is very much relying on the connection stiffness, like fixed end in a cantilever beam for façade canopy or moment joints in eave of portals. Cantilever used in supporting glass facade is sensitive to tip deflection and HSFG bolt should be used to reduce clearance movement.

2. COMMON FAILURE MODES AND BEHAVIOUR OF BOLTED CONNECTIONS

Connection failure is among the most common forms of failure in steel structures, and the remaining two are buckling or instability and material quality.

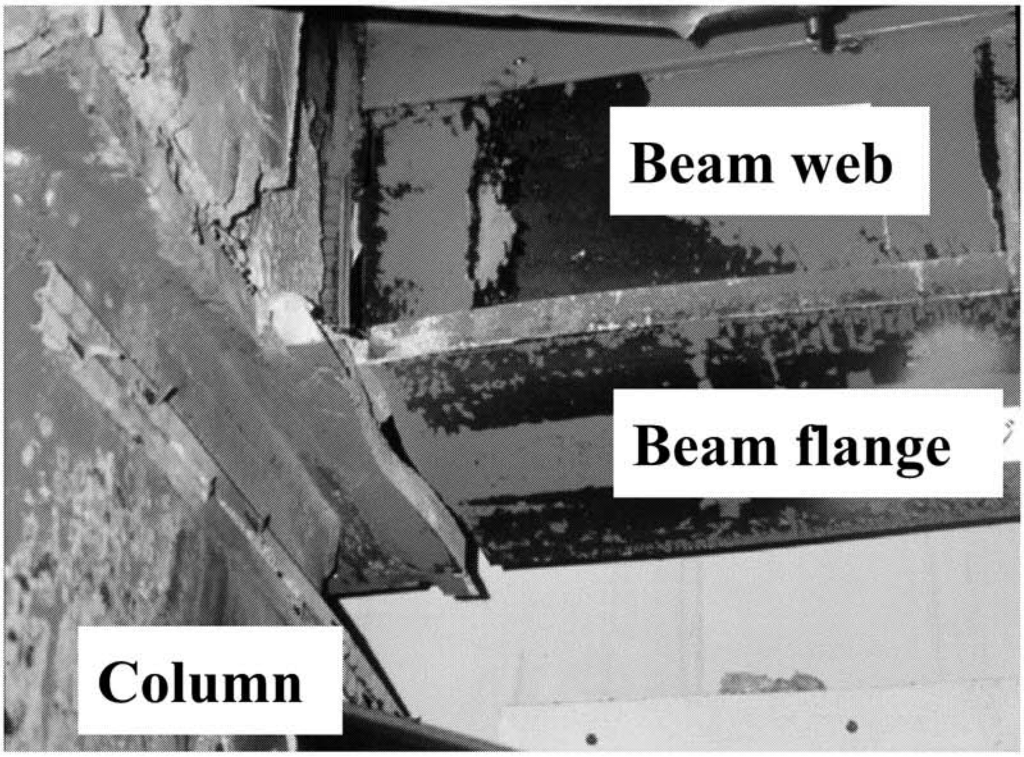

Figure 1 Failure of bolt connection in bracing members

The failure mechanism for beam-to-column and beam-to-beam connections is indicated in Figure 1.

The extended end plate connection showing the behaviour of a bolt group is demonstrated in Figure 1. This connection is used to transmit the vertical load and moment from beam to column. The point of rotation is assumed at the base of bottom flange of beam member as shown in the Figure. The corresponding deformations at the connection are caused by the loadings of moment and shear transferred from beam member so as to induce different structural effects, which are also tabulated in Table 1.

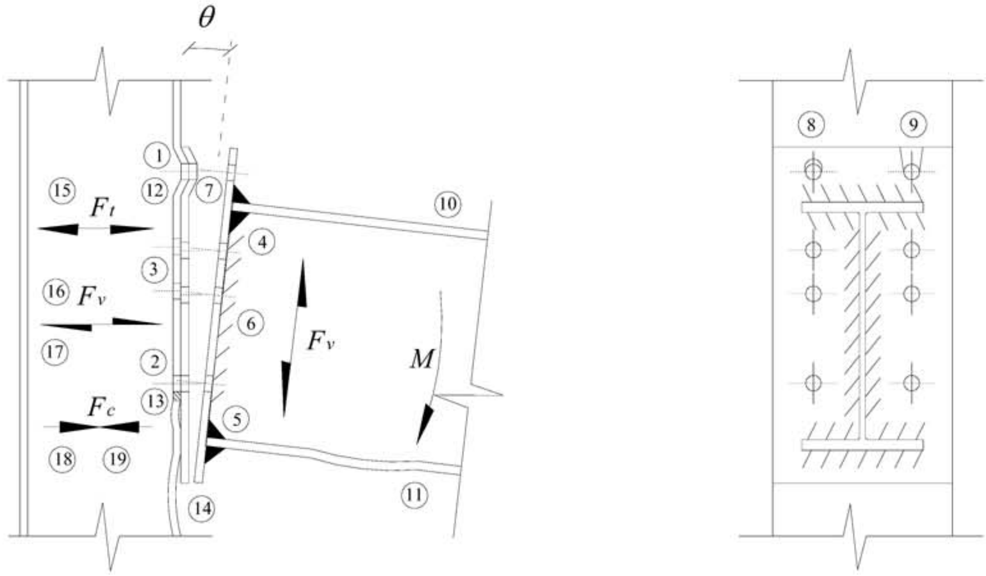

Figure 2 Behaviour of different components at beam column connection

Table 1 Structural failures at beam column connection

Components at connection	Notations	Structural effects
Bolt	①	Yielding due to tension
	②	Yielding due to vertical bearing
	③	Shear failure
Weld	④	Prying force due to bending
	⑤	Tear off failure
	⑥	Compression failure
End plate	⑦	Shear out failure
	⑧	Yielding due to vertical bearing
	⑨	Shear out failure
Flange of beam	⑩	Yielding or failure due to tension on top or bottom flange
	⑪	Local buckling on bottom flange
Flange of column	⑫	Bending
	⑬	Yielding due to shear and compression
	⑭	Local buckling due to vertical load
Web of column	⑮	Web crushing due to tension
	⑯	Yielding due to shear
	⑰	Shear web buckling
	⑱	Web crushing due to compression
	⑲	Web cripling

In Table 1 read in conjunction with Figure 2, there can be 19 principal failure modes at the connection to be checked for the components of bolt, weld, end plate, beam member and column. In this paper, the behaviour of components of bolt, weld or end plate is studied for design. Other structural components, such as column and beam, related to the behaviour at connection should be referred to other relevant parts in the code. The interaction effects on bolt and weld are neglected in Table 1 but their interactive use to share a load is not generally acceptable because of their different ductility performance. In general, the behaviour of these components at connection is principally similar. In addition, some local effects are also ignored in Table 1, which made connection behaviour more complex and variable, including the geometric imperfections arising from welding distortion and misalignment of clearance and residual stress and strain due to lack of fits and welding shrinkage. These local effects are covered by material ductility in connections. Therefore, the structural design can be carried out according to the behaviour of connection or failure mode at connection as listed in Table 1.

This paper outlines 2 new elements, among others, in the Code of Practice for Structural Uses of Steel 2005 Hong Kong. Namely they are the prying force in bolt design and in block shear failure.

3. PRYING FORCE IN BOLTED CONNECTION

For bolted connection subjected to external tension F_t, the flexible deformations at unstiffened plate element, such as flange or connected plate, induce additional tensile force in addition to axial force in bolt F_{bt}. This additional tensile force is termed as the prying force Q as shown in Figure 2 due to deflection of unstiffened plate component. The prying force Q develops because unstiffened plate of member is in contact with the connection. The contact area in compression and at the end of unstiffened plate as shown in Figure 2 produces an additional force in bolt.

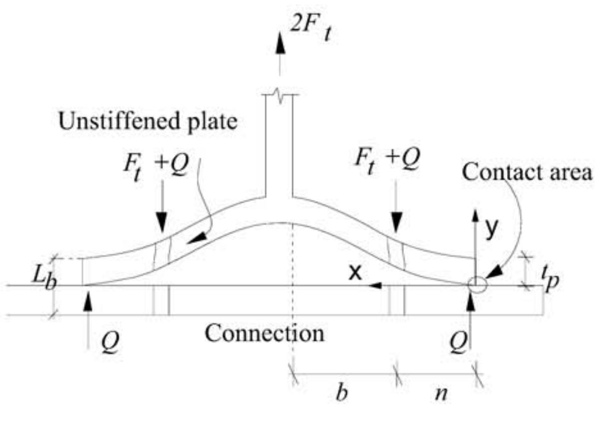

Figure 2 Prying force on bolt due to flexural deformation on unstiffened plate

The prying force for ordinary bolt is denoted as "Q" in Equation 1. Considering symmetrical half of the connection, the differential equilibrium equation is given by,

$$EI\frac{d^2y}{dx^2}+Qx-(F_t+Q)(x-n\ )=0 \qquad \text{(Eqn 1)}$$

Making use of the boundary condition as zero deflection at x=0 and zero rotation at x=b+n, the deflection at bolt location, δ_b, is given by,

$$\delta_b=\frac{1}{EI}\left[-\frac{n^3Q}{3}+n\left(\frac{nQ}{2}(n+2b)-F_t\frac{b^2}{2}\right)\right] \qquad \text{(Eqn 2)}$$

in which Q is the prying force shown in Figure 2, EI is the flexural constant of the connecting plate, F_t is half of the applied tension and b and n are the distance between the bolt and the applied tension and the edge of plate for the prying force as shown in Figure 2.

This expression is obtained from the assumption that of elastic behaviour of the plate and bolts. And the final force in the bolt Q is given by

$$Q=\frac{F_t-2EI\delta_b/nb^2}{2(n/b)+\frac{2}{3}(n/b)^2} \qquad \text{(Eqn 3)}$$

F_t is the tension applied to the connection. EI is flexural constant of unstiffened plate, which I is equal to $wt^3/12$ where w and t are width of unstiffened plate in longitudinal direction and thickness of unstiffened plate, respectively. δ_b is the axial deformation on bolt given in Equation 4.

$$\delta_b=\frac{(F_t+Q)L_b}{E_bA_b} \qquad \text{(Eqn 4)}$$

in which E_b and A_b are the elastic modulus and cross section area of bolt, respectively. L_b is grip or total length of the bolt. F_t is the total tension applied to the connection bolt. Substituting δ_b into Equation 3, the prying force Q for ordinary bolt can be obtained. For HSFG bolts, the tension force should be reduced by the pre-load force in the bolt.

The final prying force Q can then be calculated as,

$$Q=\frac{F_t\left(1-\frac{2EIL_b}{nb^2E_bA_b}\right)}{2\left(\frac{n}{b}\right)+\frac{2}{3}\left(\frac{n}{b}\right)^2+\frac{2EIL_b}{nb^2E_bA_b}} \qquad \text{(Eqn 5)}$$

4. DESIGN OF ORDINARY NON-PRELOAD BOLTS

In simple design where all joints are pinned, connections are required to be designed to take direct forces only and moment are not considered. In other cases moments are unavoidable due to eccentricities of connections, the effect of moment should be considered in finding of bolt forces. The pinned connections should be detailed to allow full rotational ductility.

In design for moment frames where full rotational continuity at connections is assumed, shear, axial forces and moments between members are needed for consideration in finding the forces in bolts. Further, detailing should attempt to provide adequate stiffness at joints. HSFG bolts are recommended when straight control of slipping in joints is required.

In design for semi-rigid connections, partial continuity is assumed between members and connections are required to have adequate strength with sufficient rotational capacity. The moment-rotation characteristics of the connection details should be consistently used both in the analysis of the framework and the design of the connections. At present, due to the lack of data in connection stiffness, semi-rigid connection design is uncommon, but it can be included in design and analysis directly and easily when the moment vs. rotation curve is available.

In the code, when the spacing of bolts is small and less than 0.55 of the width of the connection such that the induced moment is not large, the prying force can be considered by reducing P_t to $0.8P_t$ in which P_t is the design tensile strength of the bolt.

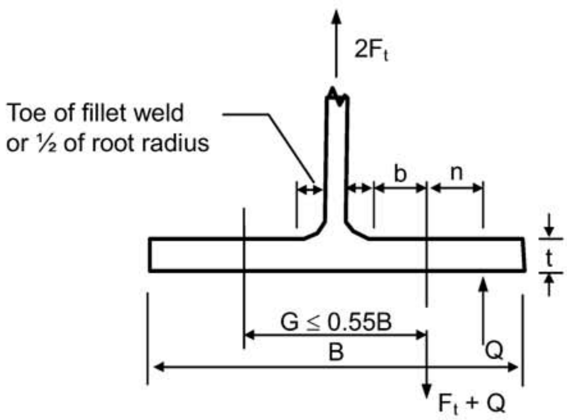

Figure 3 Prying force in bolts

When a bolt is to transfer both shear and tension, the interaction effect of the bolt should be checked, in addition to the separated satisfaction in tension and shear capacity. The additional interaction check can be carried out as follows for the case of no explicit consideration of prying force.

$$\frac{F_s}{P_s}+\frac{F_t}{P_{nom}}\leq 1.4 \qquad \text{(Eqn 6)}$$

in which F_s is shear force on each bolt and F_t is tension force in the bolt. If the prying force Q is calculated explicitly, the condition of interaction effect is given by,

$$\frac{F_s}{P_s}+\frac{F_{tot}}{P_t}\leq 1.4 \qquad \text{(Eqn 7)}$$

in which F_{tot} is the sum of applied tension force without prying and the prying force.

If not comfortable on the assumption such as location of prying force Q which in fact should be a non-uniform distributed load rather than a point load, use the finite element method. Below is a shot on a project in Hong Kong and the approach was found to be both feasible and accurate.

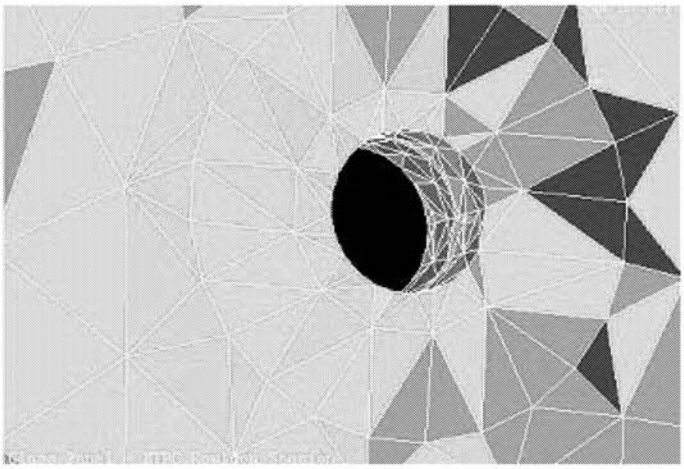

Figure 4 Study of stress around bolt location allowing for prying action as well as stress concentration

5. BLOCK SHEAR

The second new consideration of design of bolts will be on the calculation of a new but rather uncommon failure mode, the block shear failure.

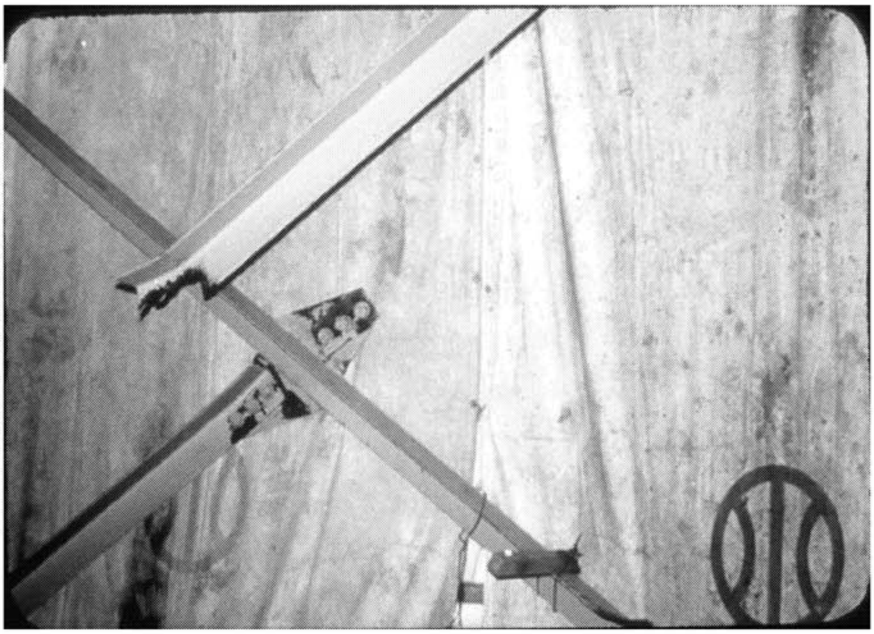

Figure 5 Failure of bolted connection due to the block shear action

Apart from the shear failure occurring on a bolt, the block shear failure of a group of bolts is required to be checked. The shear failure surface will be constructed by assuming the minimum length for shearing off of the bolt group shown in Figure 4 and the checking eliminates the failure of tearing off in thin plates at connections.

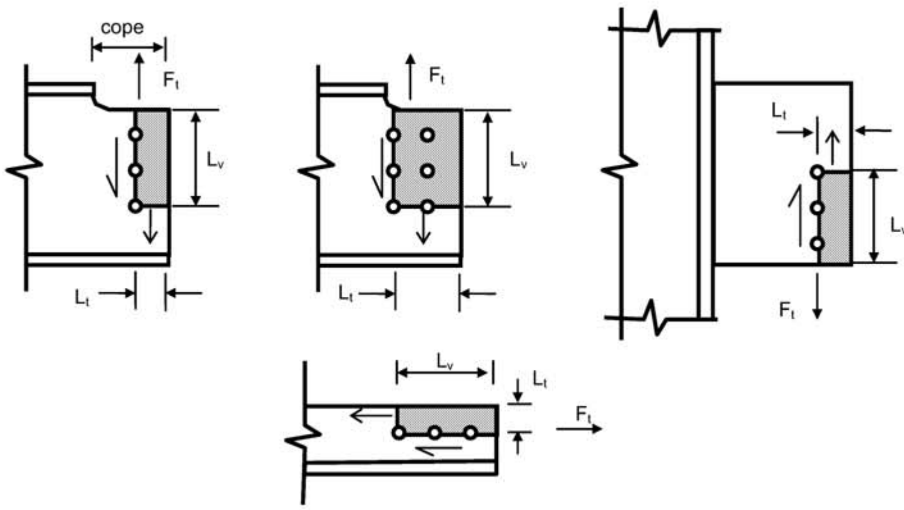

Figure 6 Block shear failure through on a group of bolt holes

The combined block shear capacity for both the shear and tension edges or faces in a shear joint shown in Figure is given by,

$$P_r = 0.6 p_y t \left[L_v + K_e \left(L_t - kD_t \right) \right] \qquad \text{(Eqn 8)}$$

in which p_y and t are the design strength and thickness of web of beam or bracket, respectively. Lv and Lt are respectively the length of shear face and tension face shown in Figure 4. K_e is the area coefficient in *Clause 9.3.4.4* of the Code. k and D_t are effective net area coefficient and diameter of bolt hole along tension face, respectively. k is a factor equal to 0.5 for single row of bolts and to 2.5 for double row of bolts.

If block shear check is not satisfactory, increasing the plate thickness, welding of an additional plate to increase the thickness or increasing the length of the failure surface can be considered.

6. REFERENCES

1. Code of Practice for Structural Uses of Steel Hong Kong 2005, Buildings Department, Hong Kong SAR Government.

PROTECTIVE STRUCTURAL SEALANT GLAZING A PROVEN WINDOW FIXING & BONDING TECHNOLOGY FOR PROTECTING BUILDING TENANTS & PUBLIC SAFETY

Mr. J. Ma

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Protective Structural Sealant Glazing

A Proven Window Fixing & Bonding Technology for Protecting Building Tenants & Public Safety

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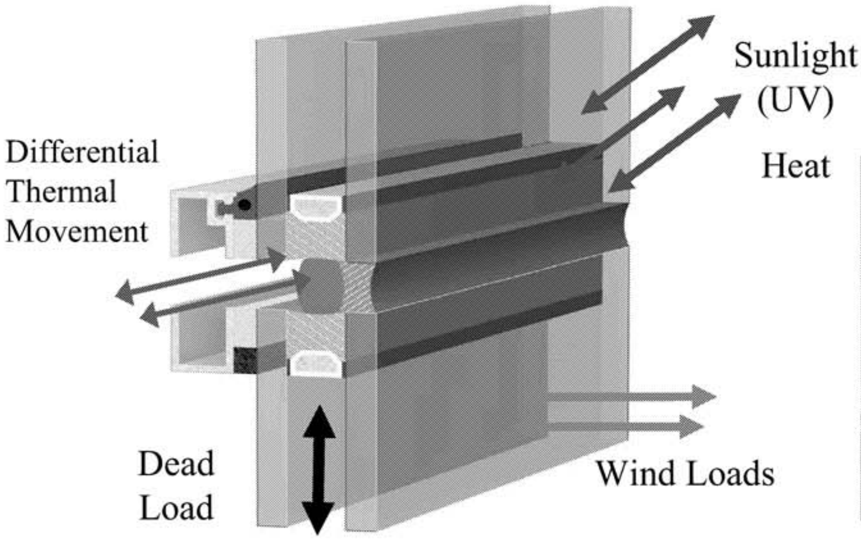
Content

- Structural Silicone Sealant Glazing – A Proven Glass Bonding Technology
- What is Protective Glazing?
- How the Silicone Sealant Contribute to Protective Glazing?
- Project Review
- What Technical Support You Need From Sealant Supplier?
- Summary

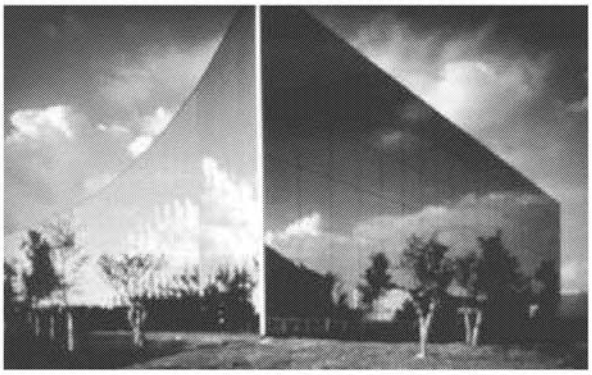
Structural Silicone Glazing (SSG)

- **Load bearing bonding between glass and supporting metal frame using structural silicone sealant**



-Bonding must be dimensioned according to (thermal, live & dead) load requirements

-SSG sealant carries loads and compensates movements



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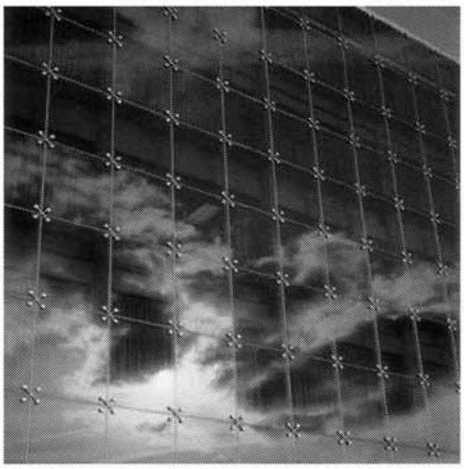
- They remain stable, flexible from -40 to 150°C to accommodate thermal expansion



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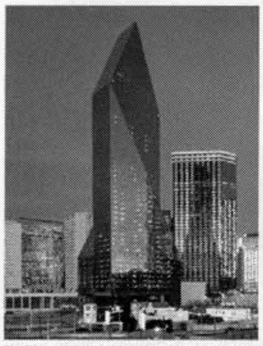
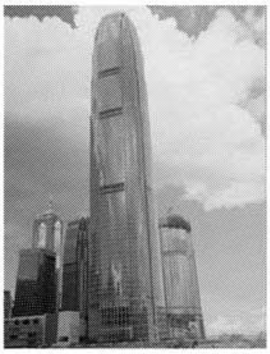

The 720-foot Allied Bank Tower is the tallest building that uses 4-sided structural silicone glazing.



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What Is Protective Glazing?

- Window systems that protect building tenants and passersby from broken glass from:
  - Severe weather
  - Falling glass
  - Bomb Blast
  - Earthquakes
  - Criminal activity

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What Is Protective Glazing?

- A Window system that uses a protective laminate, a framing system and an attachment system to protect people from flying glass:
  - Laminated glass
  - SentryGlass, Sentry Glass Plus
  - Glass-clad polycarbonate
  - Retrofit window films (3M, Solutia)

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Severe Weather

- Typhoon and other high-wind events can cause tremendous damage

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Falling Glass – A "Life Safety" Issue

- Poor window design or glazing failure may allow glass to spontaneously fall from its frame


仿落雹飛瀉 濺傷兩嫲孫

青衣城爆玻璃 百客逃生

商場拆走同類玻璃

Falling Glass – A "Life Safety" Issue


一日八宗墮窗時序表

8/28/2005

8/29/2005

朗豪坊通天梯爆玻璃雨

九樓狂灑遊人驚慄逃避

幸無傷亡

五月爆圍欄玻璃傷四客

Bomb Blasts – The Nairobi Experience

- When a truck bomb exploded outside the U.S. embassy in Nairobi, 24 people were killed in the embassy compound
- Across the street, 5000 people were injured by flying glass and 186 were killed

Criminal Activity – The Smash-and-Grab

- In 2005, burglars broke through the windows of a Gucci Taipei retail store and stole more than NT$2MM goods within minutes


民生報
消費資訊站
Gucci遭竊 上架兩天 秋冬包被偷光
小偷只偷鞋、包 不拿高價表、皮衣 旗艦店損失數百萬
清晨5點遭竊

Why use a Silicone?

- Longest lasting sealant available
  - Durabilty is proven in SSG and field applications
  - Water leakage can cause anchor corrosion
- Small changes in modulus with strain rate
- Small changes in modulus with temperature
- Excellent dampening and shock absorbing capabilities
- High strength formulations available
- Excellent and durable adhesion to glass and metal frame

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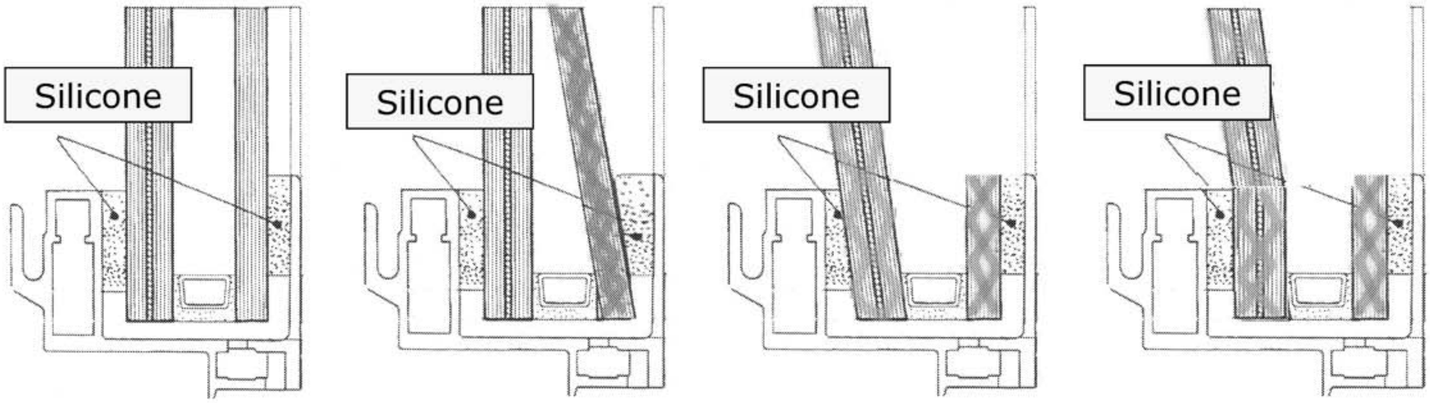
Conventional Window System with Structural Silicone



Before Explosion

During blast, outer glass pane breaks

Inner laminated glass first absorbs shock of the blast then breaks. The silicone sealant "anchors" the laminated glass in the frame protecting occupants of the building

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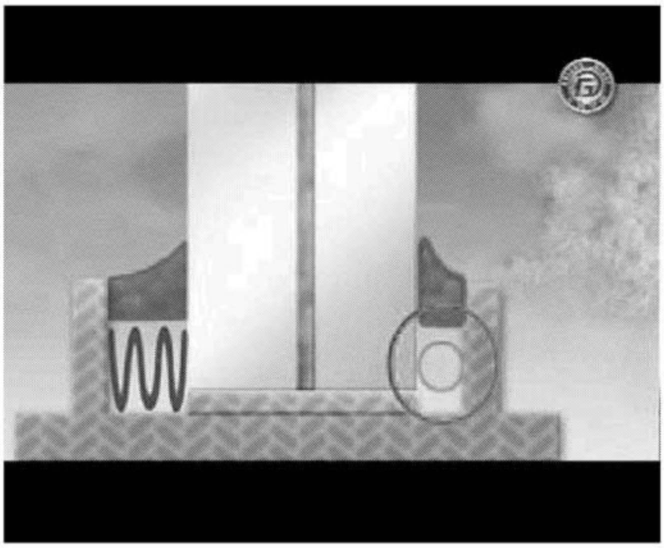
Structurally Adhered Mechanically Fastened

- Structural silicone acts in shear/tension in a blast event
- Structural silicone must rebound and not lose its resiliency
- Structural silicone must keep the water and environmental factors away from critical anchorages



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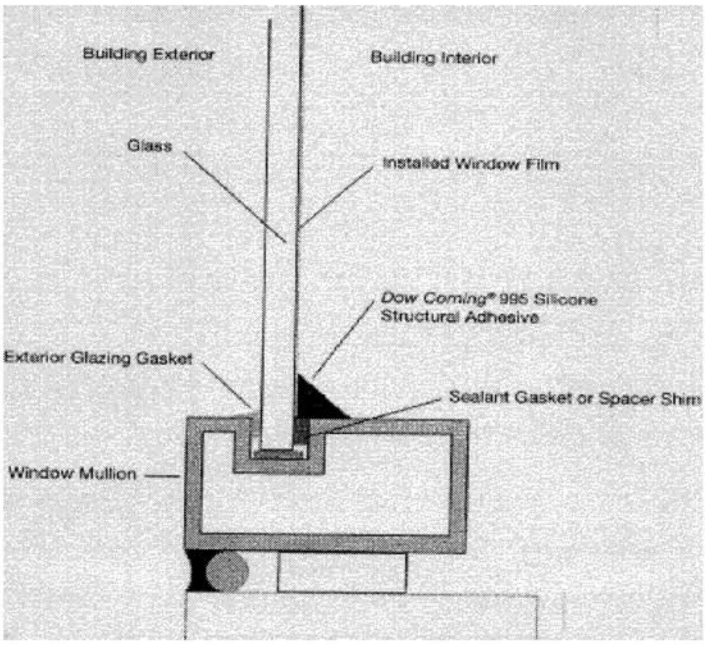
Protective Glazing Film System

Discovery Channel Video

NT

Protective Glazing Film System

- Newer approach for “low blast”
- SG adhesion between DC 995 and a Protective polyester Film
- Glass is held together when broken
- Easy and economic technique
- Improved protective glazing for window systems
- General guideline



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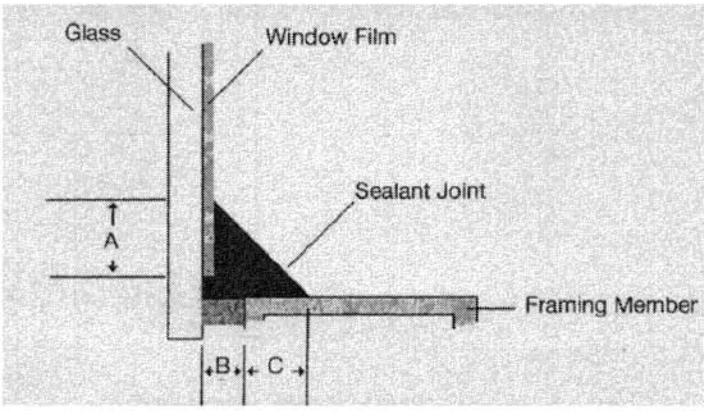
Protective Glazing Film System

Triangular joint, most common acceptable



- Examples

$A_{min} = C_{min} = 6$ mm ; B = 6 mm

$A_{des} = C_{des} = 10 – 13$ mm

Which Dow Corning Silicones?

- Dow Corning® 995 Structural Silicone Sealant
  - For both shop and field glazing
  - Passed bomb test with 3M/sloutia protective films system
- Dow Corning® 993 Structural Silicone Sealant
  - For shop structural sealant glazing only
  - Passed bomb test in UK 1997

Field Explosion Test – UK

- DC 993 test on Oct 14th 1997
- The test had been performed in an independent laboratory
- 12kg TNT 6.6 m distance away Bomb Blast on a 4-sided Silicone Structural Glazing.
- The results confirmed that SSG systems could resist both the amplitude and the high speed of the Blast.

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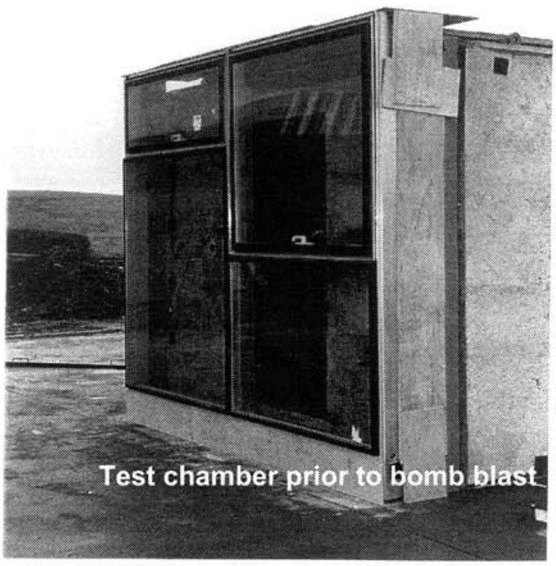
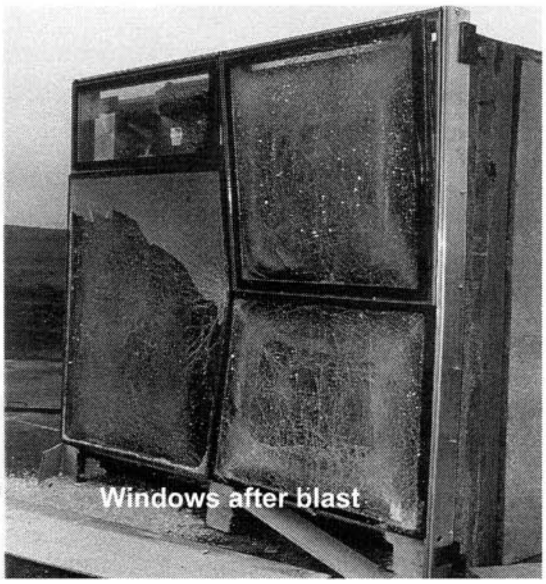
Field Explosion Test – UK
Structural Sealant glazing Facade

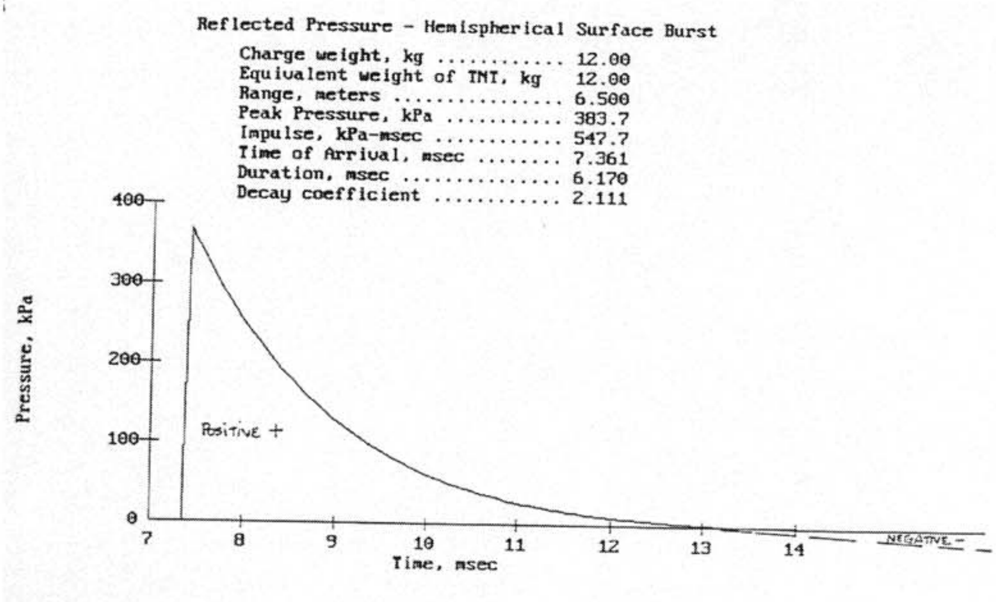
- Only high-performance silicone sealants keep broken glass in window frames under high, sudden bomb blast stress


Test chamber prior to bomb blast


Windows after blast

Field explosion test

- Parameters:



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Field Explosion Test – UK Structural Sealant glazing Facade

Outcomes:

- The design has to specify:
  - the composition of the glass
  - the value Bomb Blast to be absorbed and how to absorb the Blast
    - one laminated glass
    - one IG unit
      - tempered + laminated
      - laminated + laminated
    - one additional external structural skin to the facade to reduce the loads onto the facade itself

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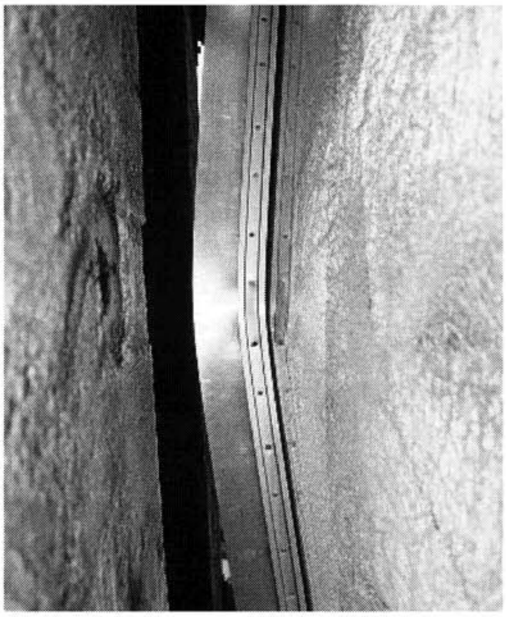
Field Explosion Test – UK
Structural Sealant glazing Facade

Outcomes:

- Main concepts:
  - the structural silicone has to be dimensioned so that it doesn't break, until the end the first impulse or, at least, until the breakage of the tempered glass units in order to avoid that the glass could fly out entire.
  - the laminated glass must have a sufficient number of PVB layers to withstand the blast



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Bishops-gate 99 / London

- Architect GMW Partnership - 1996.
- Renovation of a concrete based building.
- 15 000 sqm 4 sided Structural glazing (DC 993), with Low-E Glass, thermal break and high acoustic performances.
- The facade is designed to resist a bomb blast of 8000 Pa.
- Fast panel installation using the "Top-Down" concept by the interior of the Building.

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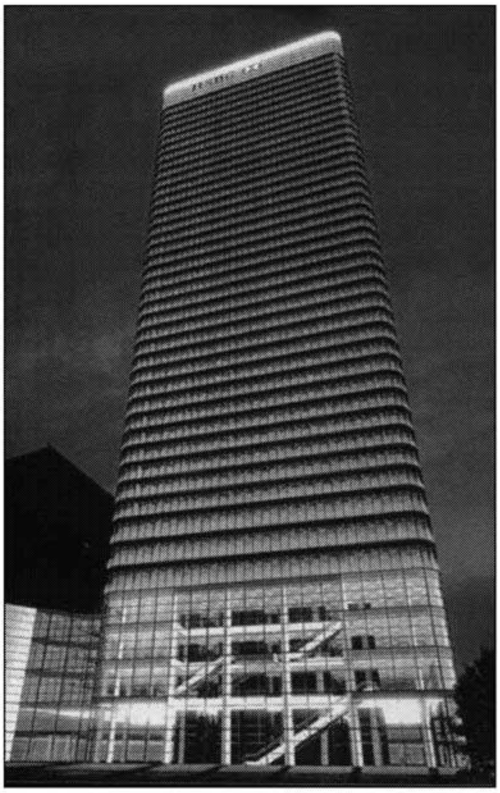
DS-2 London

- Architect: Fosters & Partners
- New Design – Canary Wharf
- >15 000 m^2 4-sided Structural Glazing with high performance Silicone bonding
- Bomb blast mitigating window design on podium (first 5 stories)



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DOW CORNING

Projects Review

A Proven Window Fixing/Bonding Technology for Protecting Building Tenants & Public Safety

Marathon Plaza - San Francisco

- City's largest concrete commercial building.
- 2-sided Structural Glazing system.
- Faced in 1989 by California's worst earthquake since 1906.
- Successfully passed the "test". Not a single lite was lost.

- Neighboring building had windows crack and fall out. Nearby Embarcado Freeway suffered massive damage.

Atria West office building - Los Angeles

- Structural Facade + Atrium.
- 2-sided Structural Glazing system.
- Faced by severe earthquake in 1994, 20 km from the epicentre.
- Successfully passed the "test". None of its hundreds of glass lites were cracked or lost.

Exchange Square

Hong Kong

- **Construction Date:**
  - **1984**
- **Key Players:**
  - **Architect: Palmer and Turner**
  - **Curtainwall contractor: Gartner and Builders Federal HK**
  - **Consultant: Victor Mahler**
- **Environment:**
  - **Frequent heavy rain and typhoons**
  - **Tropical Heat and Humidity**

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Exchange Square

Hong Kong

- **Type: 2-sided unitized factory glazing**
- **Building Size: Twin 200-m towers**
- **Structural Sealant:**
  - ***Dow Corning®* 795 Silicone Building Sealant**
  - ***Dow Corning®* 983 Silicone Glazing Adhesive/Sealant**
- **Curtainwall Details:**
  - **Sealant design strength: 138 kPa**
  - **Lite#1 –Vision**
    - **Sealant bite: 40 mm**
    - **Dimensions: 1600 x 1280 mm**
    - **Windload: 5.27 kPa**
  - **Lite #2 – Spandrel**
    - **bite: 40 mm**
    - **Dimensions: 770 x 1280 mm**
    - **Windload: 5.3 kPa**
  - 10,000 factory-glazed units including glass and granite spandrel

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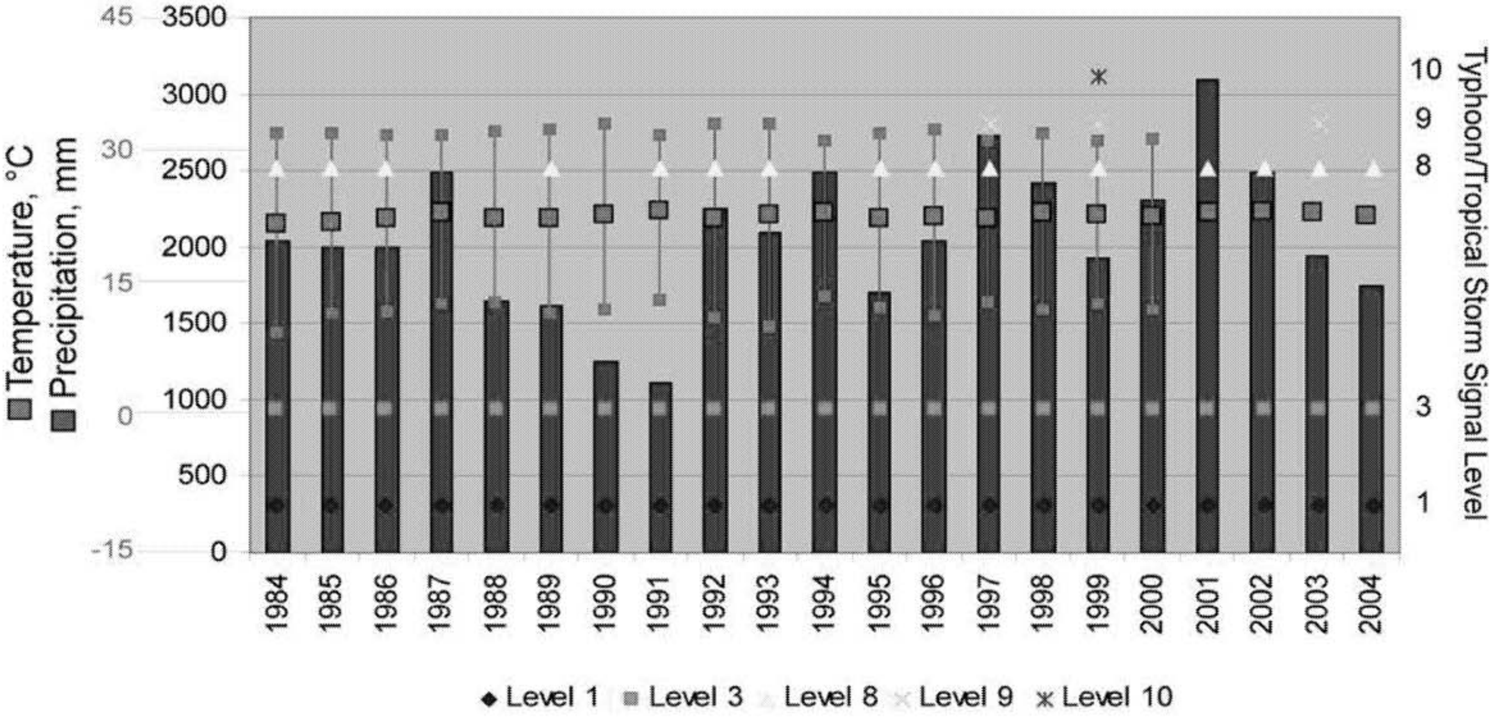
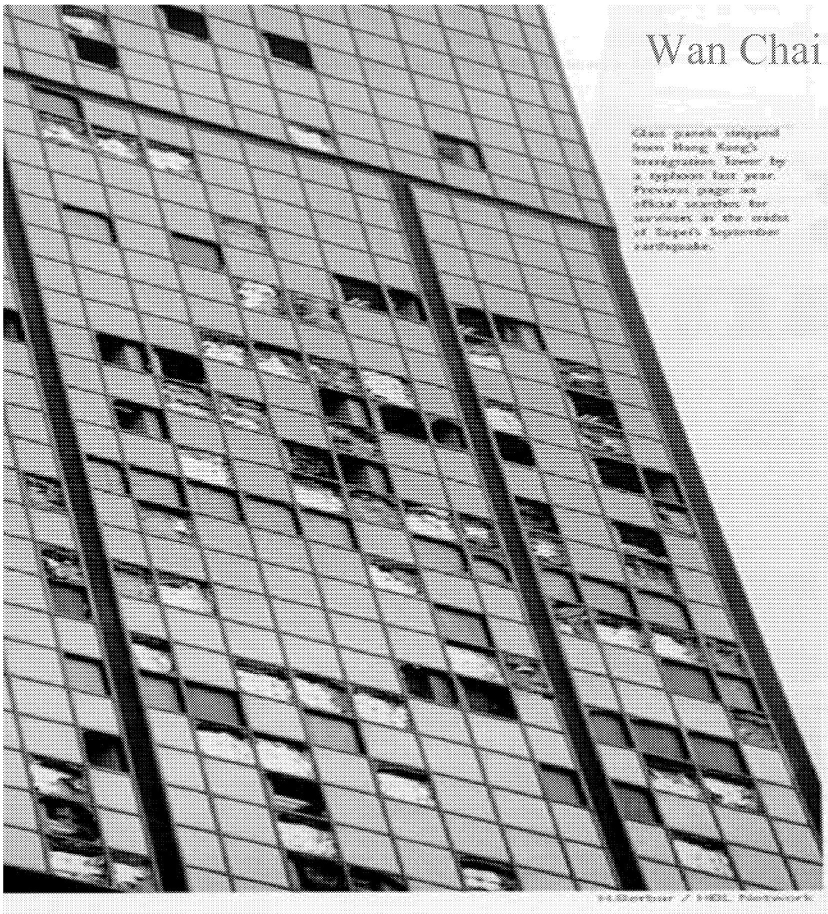
Exchange Square

Hong Kong



Dow Corning PROPRIETARY

Wan Chai Government – Glass Fallen Down



Glass panels stripped from Hong Kong's Immigration Tower by a typhoon last year. Previous page: an official searches for survivors in the midst of Taipei's September earthquake.

H.Barbar / HBL Network

for wind-loading, it also specialises in design detailing for protection from earthquakes – work the company has done on projects in Taipei, LA and New Zealand.

Wymond says while developers meet the Hong Kong safety standard for typhoon loading, most feel it unnecessary to go a step beyond that, as damage is usually covered by insurance. He says more should be done in terms of using greater impact resistant materials, such as laminated glass.

Most tall buildings in Hong Kong are designed to sway about half a metre for every 250 metres of height – the standard wind-loading regulation. While the flexibility provides some protection from minor tremors (though tremors and high winds affect buildings in different ways), an earthquake of 5.6 or above could have disastrous consequences on buildings not adequately designed or built.

He says while nobody expected a big earthquake to hit Newcastle, the same thing could just as easily hit Hong Kong.

"Before that quake, the severity of an earthquake [on a large continental plate] was perceived to be quite low, but after that people realised you can get quite large earthquakes within a big plate," Wymond says.

"The Newcastle quake woke up the industry in Australia, and maybe some of it was a sudden reaction, maybe slightly an overreaction, but it is still erring on the side of caution, which is good."

"Erring on the side of caution is definitely not happening up here, with a few exceptions."

The building assets in Hong Kong were estimated to be worth more than HK$4000 billion several years ago, so even damage to a tiny proportion would be expensive.

Even though the centres of two of the stronger earthquakes recorded in Hong Kong were to the south of the island, mainland China is considered to be at greater risk of seismic activity than the SAR. Therefore, the PRC has seismic regulations, but Hong Kong does not.

Says Wymond: "As soon as you go across the border from Hong Kong, the Chinese design for earthquakes, and yet we don't do it here, which is a little bit strange – and maybe an oversight."

Only a handful of buildings in Hong Kong are designed to resist earthquakes, which include the Meinhardt and LRA-designed Bank of China, the Cheung Kong Centre and Arup-designed Hopewell Centre, and the ICC Building. A notional level of seismic loading is also built in to some hospitals and roadwork infrastructure.

Apart from China, buildings in Taiwan, the Philippines and Japan are also designed with a stronger seismic loading due to the fault stretching down through these countries.

While Ove Arup's Vesey states there is no cause for alarm of an earthquake just around the corner, he says seismic regulations are an issue under review in the ►►

CONNECTIONS 18

Proven Performance in Real Blast

"Bomb in Madrid City Centrum" July 2000

A 20 kg Bomb exploded between two buildings in the city Centrum of Madrid

Dow Corning PROPRIETARY

Proven Performance in Real Blast

"Bomb in Madrid City Centrum"

- Since this façade (1993) was not designed to be bomb blast resistant, and was mainly an architectural cladding to mask brick walls, had been registered few damages inside the building … fortunately, we have to say.
- The loads, unknown but certainly much stronger than the design wind, broke glass and frames…
- … but were not sufficiently high to break the structural glazing silicone joints.

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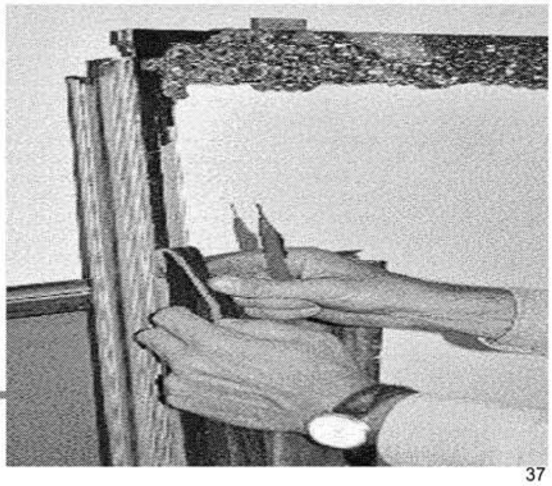
Proven Performance in Real Blast

"Bomb in Madrid City Centrum"

- The structural glazing silicones (DC 895, DC 983) did not break and kept their adhesion while glass was breaking.
- The glass broke before transferring too high stresses.



World Trade Center

Collapsed in September 11 2001 by Terrorist Attacked

911 World Trade Center
Hilton Millennium adjacent to the World Trade Center, NY, USA

The Pentagon

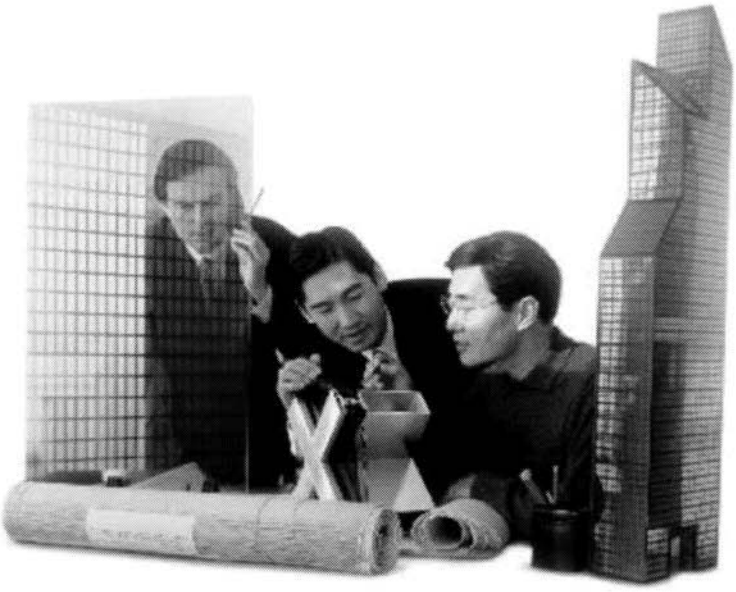
Installed Protective Glazing Window System several months before 911 Attack.

Windows were remained after suicide airplant crush attacked

Pentagon Washington D.C. USA
Windows do not fall out or damage during 911 attack

The Dow Corning Support You Need

- The technical support you need to design, specify and ensure quality installation of non-structural, structural and protective glazing window systems



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The Dow Corning Support You Need

Dow Corning offers Project Technical Services

- **Review Shop Drawings**
- **Adhesion Testing (ASTM C794)**
- **Compatibility Testing (ASTM C1087)**
- **Bomb Blast Technical Data/information sharing**
- Regular site/factory visits
- Provide training and general technical support
- Deglazing – quality control

International Standard for Protective Glazing System

BS EN 1354:2001 Glass in Building – Security Glazing – Testing and Classification of Resistance Against Explosion Pressure. A "Material" Shock tube test method, 50, 100, 150 and 200 kPa Tests

BS EN 13123-1:2001 Window, Doors and Shutters – Explosion Resistance – Requirements and Classisfication – Part 1 : Shock Tube, 50, 100, 150 and 200 kPa tests.

BS EN 13123-2: 2004 Window, Doors and Shutters – Explosion Resistance – Requirements and Classisfication – Part 2 : Range Test.

CEN/TC 129, EN 13541: **2001** "Security Glazing – Testing and Classification of Resistance Against Explosion"

CEN/TC 33, EN 13124: **2001** "Windows, Doors and Shutters – Explosion Resistance"

ASTM C 1564:2004 Standard Guide for Use of Silicone Sealants for Protective Glazing Systems.

ASTM F 1642: **2004**Test Method for Glazing and Glazing Systems Subject to Airblast Loadings

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Technical Articles Relate to Protective Glazing System/Design

Design for Glazing Protection Against Terrorist Attacks: Sergio De Gaetano, Whitbybird, UK, Glass Processing days 2005 P. 505-508

International Standard for Arena Testing of Blast Resistant Glazing: Robert A. C Stephens & Nicholas F Johnson, UK Home Office Scientific Development Branch, Glass Processing Days 2005 P.490-493

Selection of Glazing Materials for Blast Protection: Ryan A. M. Sukhram, Arup Security Consulting UK Glass Processing days 2005 P.544-546

The Use of Silicone Sealants in Protective Glazing Applications: Jean-Paul Hautekeer etc Dow Corning Corporation Glass Processing days 2001

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Summary

- The Structural Silicone Glazing is a very attractive and proven technique for Bomb Blast and other extreme conditions (Earthquake, Typhoon) resistant façade, that start being well known
- Protective glazing designs protect buildings but more importantly protect the **people** in the building
- System (glass, mullions, ...) needs to be properly designed; for that has a great importance the partnership between all parties: designer, curtain wall manufacturer and silicone producer.

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Summary - Why Dow Corning Silicones?

Only Dow Corning Silicone Sealants:

- Had decades of experiences in structural sealant glazing applications and long experience with protective glazing system manufacturer, designers and installers
- A proven effective in actual bomb blast testing and performing satisfactory in many actual projects
- Have been evaluated as the best attachment product for a retrofit film window system

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The Dow Corning Support You Need

It’s a dangerous world.

Dow Corning can help you make it a little bit safer.


退休金沒了 改當炸彈客

南美玻利維亞一名四十七歲退休礦工，上月三十日為索討退休金，在拉巴斯國會大廈引爆纏在身上的炸藥，當場把自己炸死，還造成一名安全首長及一名警官罹難，另外七人受傷。圖為警方拆除自殺炸彈客放置於國會的一枚炸彈，引爆當時一名警官趕快摀起耳朵找掩護。 法新社

DESIGN AND FABRICATION OF WELDED CONNECTIONS

Ir Dr W T CHAN[1]

Buildings Department, HKSAR Government

ABSTRACT

In the construction industry, steel structures usually involve various types of connection, viz. welded connection, bolted connection and adhesive-bonded connection. Welding in this context is the joining of parts to constitute a continuous metallic bond.

From an engineering point of view, welding transforms the mechanical properties of metal and hence should only be used under carefully controlled conditions. If inevitable, welding should best be conducted under controlled shop environment while bolting is more suitable to be carried out on site.

The paper aims to present the design aspects of commonly encountered welded connections while some preparatory measures and working details are recommended in design and fabrication stages so as to mitigate the adverse effects arising there from.

Keywords: Welded connections, stress concentration, butt weld, fillet weld, full penetration butt weld, partial penetration butt weld, plug weld, slot weld, flare groove weld, compound weld, effective weld length, throat thickness, welding symbols.

1. INTRODUCTION

Joints are fabricated mechanically using welding, bolts, rivets, or adhesives. Welding in this context is the joining of parts to constitute a continuous metallic bond.

Welding can bring about an unfavourable transformation in the mechanical properties of metal, which may undermine the design strengths and other mechanical properties.

In general, welded connections are subject to distortion and high residual tensile stress, and may be susceptible to defects such as lamellar tearing and hydrogen embrittlement, etc. These phenomena can be mitigated and avoided if the welded connections are properly designed for and fabricated correctly.

---

[1] Senior Structural Engineer, Buildings Department, HKSAR Government
Chairman, Joining and Welding Group, Hong Kong Institution of Steel Construction

Firstly, the paper aims to depict a general review on the essential engineering properties of welds followed by different types of welded connection.

Secondly, the structural design of welded connections is exemplified with popular methods and worked examples. Recommendations on the determination of weld leg length at unstiffened flanges and weld throat thickness are given.

Thirdly, as welded connections are subject to lamellar tearing and hydrogen embrittlement, these phenomena can be mitigated and avoided if the welded connections are properly designed for and fabricated. Recommendations on the end returns of welded geometry, transition geometries of joining plates of different thicknesses, avoidance of distortion, relief of high residual tensile stress and removal of slag inclusions are given so as to enlighten the readers of possible measures to mitigate welded defects. Some bad practice in welding as well as conventional weld symbols are also mentioned for reference.

Fourthly, current standards on welding consumables and testing of welds versus the obsolete standards are listed in Appendix I for ease of reference. In general, the current BS EN or BS EN ISO standards are more easily complied with as compared with those obsolete BS standards since the requirements are to a certain extent relaxed. It is encouraged that the practitioners should adhere to the use of the current standards, albeit the stakeholders in local construction industry, such as the steel suppliers and HOKLAS accredited testing laboratories, may need to take some time to adapt to the transitional change.

Last but not the least, a comparison is conducted amongst the various design standards on welding. A summary is tabulated in Appendix II for ease of reference.

It is of paramount importance to control welding in the incipient stage by proper design of welded connections as well as adopting proper working details in order to safeguard the weld qualities.

2. ENGINEERING PROPERTIES OF WELDS

a) Tensile strength

The design strength (p_y) given in the materials standards relates to the longitudinal and transverse directions. In general,

$$\text{Derived design strength } p_y = \quad Ys/\gamma_{m1} \leq Us/\gamma_{m2}$$

where γ_{m1} (=1.0) & γ_{m2} (= 1.2) are the recommended minimum partial material factors applied to the yield strength and ultimate tensile strength (Us) in the Code of Practice for the Structural Use of Steel 2005 (the Code).

b) Prevention of Brittle Fracture – notch toughness

The minimum average Charpy V-notch impact test energy of 27 Joules at the required testing temperature for the strength of steel, thickness involved and minimum design service temperature as required by the structural design code should be provided in order to possess sufficient notch toughness.

c) Ductility

The minimum ductility of steel is limited to 15% elongation, which is merely used in plastic analysis requiring a higher degree of ductility for the formation of plastic moment.

d) Weldability

Steel should have appropriate chemical composition that matches with the fusion of the base metal and the filler metal without the formation of cracks and other imperfections. This is defined as the weldability of steel. Usually, the calculation of Carbon Equivalent value (CEv) is based on the empirical equation generally used in IIW.

$$CE = C + Mn/6 + (Cu+Ni)/15 + (Cr+Mo+V)/5$$

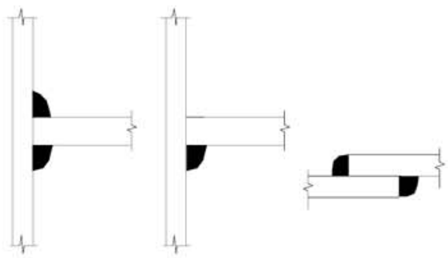
3. TYPES OF WELD

The several basic types of weld, viz.

a) Fillet weld
   i. Continuous and intermittent weld
   ii. Deep penetration fillet weld
   iii. Plug weld
   iv. Slot weld

b) Butt weld
   i. Full penetration butt weld
   ii. Partial penetration butt weld
   iii. Compound weld

c) Flare groove weld



d) Double fillet weld   Single fillet weld   Fillet weld for lap joint

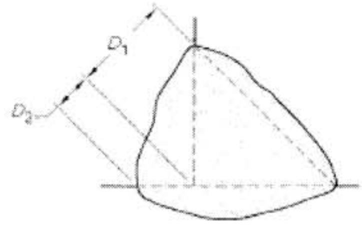


Figure 1 Different types of fillet welds

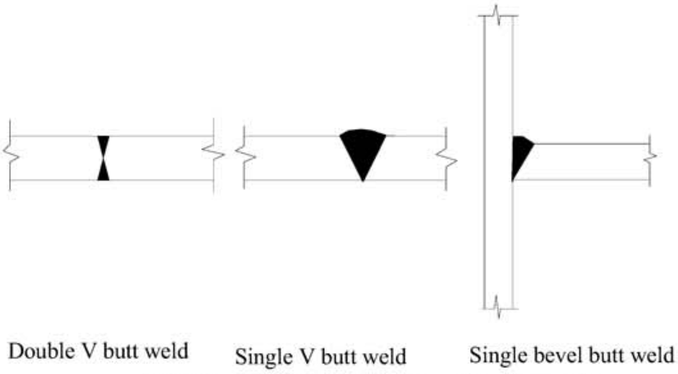
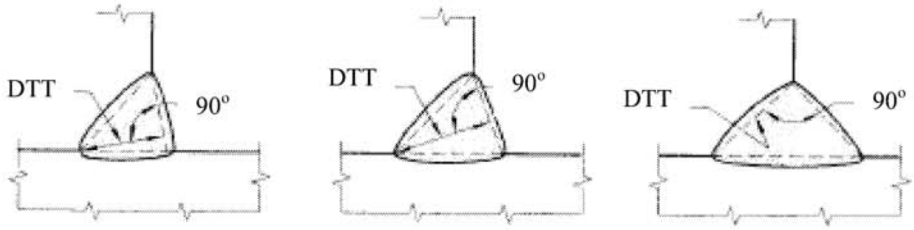
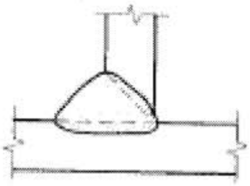


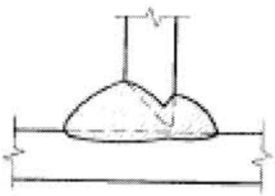
Figure 2 Different types of butt welds



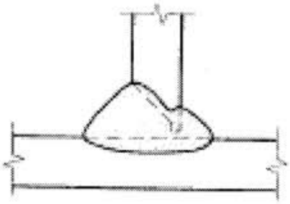
Design Throat Thickness of compound Welds with Incomplete Penetration Welds



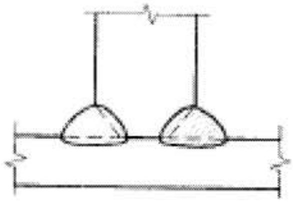
Fillet weld superimposed on
Single bevel butt-welded T-joint



Fillet weld superimposed on single bevel
Butt-welded T-joint with an additional
Fillet weld on the root side



Fillet weld superimposed on Single
J-butt-welded T-joint with an additional
fillet weld on the root side



Fillet weld superimposed on
incomplete penetration bevel weld

Figure 3 Compound Welds

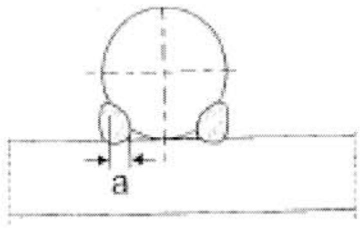


Figure 4 Effective throat thickness of flare groove welds in solid sections

4. STRUCTURAL DESIGN OF WELDED CONNECTIONS

a) Butt weld design

For welds made with matching weld metal, the strength of a full penetration butt weld in steel is the same as the parent metal. Butt welds are generally classified into two categories, viz. full penetration welds and partial penetration welds, where the strength of the weld depends on the depth of penetration of weld. Butt welds normally require 100% test and are more expensive and less common unless it is strictly necessary to ensure the welded part will not fail earlier than the parent material.

Partial penetration weld strengths are calculated on the size of their throat. It is important to be able to confirm the throat size either by inspection or by weld tests. For this reason, partial penetration welds used in tension or shear should be subject to a conservative design stress and should be treated as fillet welds if fatigue loading has to be carried.

As stipulated in the Hong Kong Code of Practice for the Structural Use of Steel 1987, the partial penetration butt weld should have a throat thickness of at least 7/8 of the thickness of the thinner plate joined. For the purpose of stress calculation and to allow for the effects of eccentricity of the weld metal relative to the parts joined, a nominal throat thickness not exceeding 5/8 of the thickness of the thinner plate joined should be taken. In the Code BS 5950-1:2000, the minimum throat thickness of a longitudinal partial penetration butt weld should be 2t, where t is the thickness of the thinner part joined.

It is hereby emphasized that for partial penetration butt welds in tension, if the centerlines of the gross and net sections are not aligned, there will be an additional bending stresses due to the bending moment caused by the force times the offset distance. This can cause a significant increase in nominal stress.

b) Fillet Weld design

The design strength of a fillet weld is calculated from its size as measured by its throat thickness. Whether a connection is in tension, compression or shear, the allowable stress given in specifications and codes is the same and varies only with the strength of weld metal or the parent metal. For normal fillet welds, the throat thickness is 0.7 times the leg length.

Intermittent fillet welds are used in some work where the loads are small. They reduce the cost of welding and distortion.

Fillet welds are relatively less expensive, whereby no preparation is required before the welding process. They are more commonly used at the re-entrant corner of two pieces of metals. During welding, the electrode bisects the angle between the two pieces of metals. The size of fillet welds is measured as the leg length in Hong Kong, United Kingdom and the United States of America. The minimum size used is generally 3mm while the common sizes are usually between 6mm and 12mm or higher for structural applications. Note that in some European countries, fillet weld sizes are specified by the throat dimension.

c) Determination of effective weld length at unstiffened flanges

Owing to the flexibility of connecting plates, the weld length should be reduced in unstiffened plate elements. When the welds connected to the unstiffened plate element of an I-, H- or a box section, a reduced effective length b_e should be used when the effect of weld is also accounted for. For a rolled I- or H-section, the effective length b_e of weld should be as follows.

$$b_e = t_c + 2r_c + 5T_c, \qquad \text{but}$$

$$b_e \le t_c + 2r_c + 5\left(\frac{T_c^2}{t_p}\right)\left(\frac{p_{yc}}{p_{yp}}\right)$$

in which t_c and T_c are the thickness of web and flange of rolled I- or H-section member, respectively, as shown in Figure 5 (a). r_c is root radius of rolled I- or H-section member. And t_p is the thickness of connected plate as shown in Figure 5 (b). p_{yc} and p_{yp} are respectively design strength of rolled I- or H-column or structural members and connected plate.

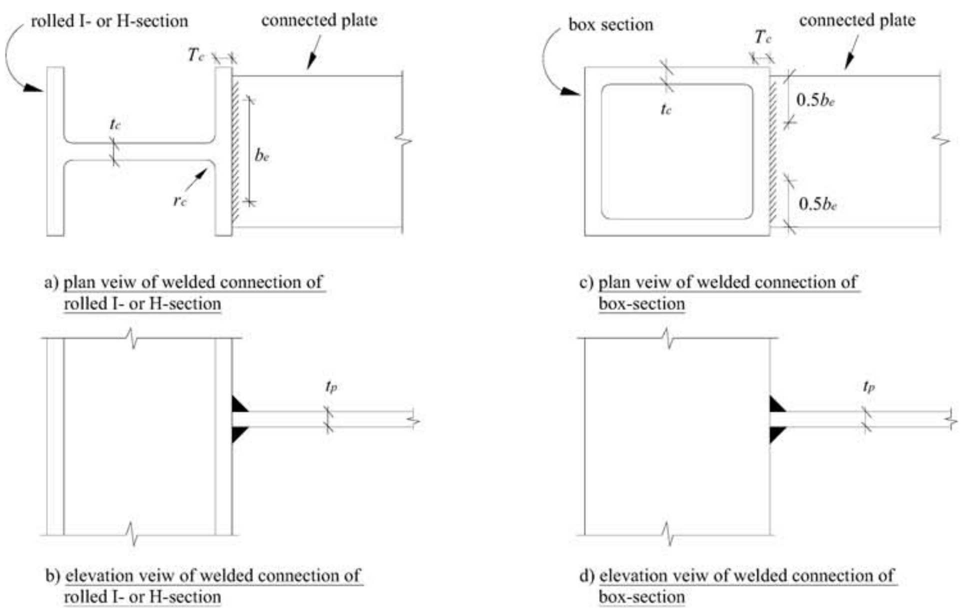


Figure 5 Effective length of weld connected to unstiffened plate element

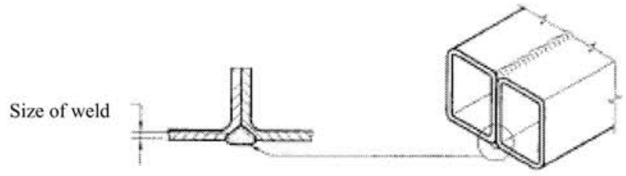
For a box section in Figure 5 (c) and 5 (d), the effective length b_e of weld is taken as,

$$b_e = 2t_c + 5T_c$$

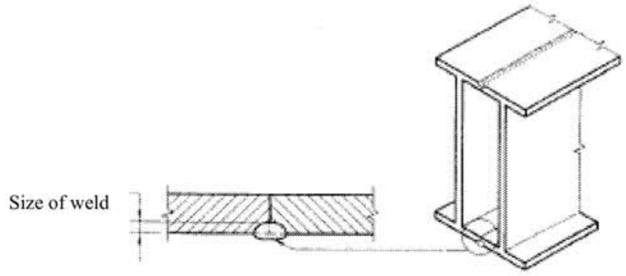
$$\text{but } \; b_e \le 2t_c + 5\left(\frac{T_c^2}{t_p}\right)\left(\frac{p_{yc}}{p_{yp}}\right)$$

where t_c and T_c are the thickness of web and flange of a box section, respectively, as shown in Figure 5 (c). t_p is the thickness of connected plate as shown in Figure 5 (d).

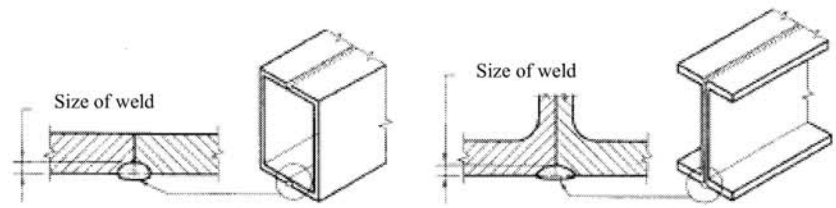
d) Determination of weld throat thickness



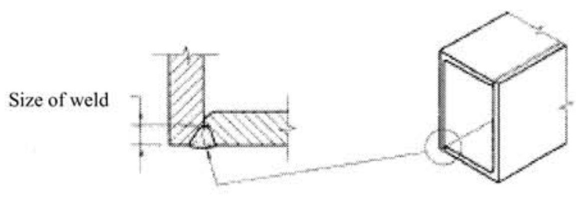
Hollow sections



Beams or columns



Channels



Angles

Figure 6 Welding of rolled sections to form built-up members

e) Calculation of welded connection capacity using Simplified method

Stresses should be calculated from the vector sum of forces from all directions divided by the weld throat area to ensure it does not exceed the design strength of weld p_w. This is a simpler but less economical approach of finding the resultant stress acting on a weld and checking of this resultant stress against the design strength of weld as, $P_L \geq F_R$

in which F_R is the vector resultant stress equal to $\sqrt{F_x^2 + F_y^2 + F_z^2}$ on the weld.

f) Calculation of fillet welded connection capacity using Directional method

The design strength p_w of weld depends on the size of weld, such as the leg length s, which is the size of fusion face on the unprepared surface of parent metal as shown in Figure 7. Moreover, the strength p_w of weld is based on the material used in the welding electrode and strength of parent metal. Further the throat thickness a, which is the perpendicular distance from inclined surface of weld to root of weld as illustrated in Figure 7, is determined from the leg length s.

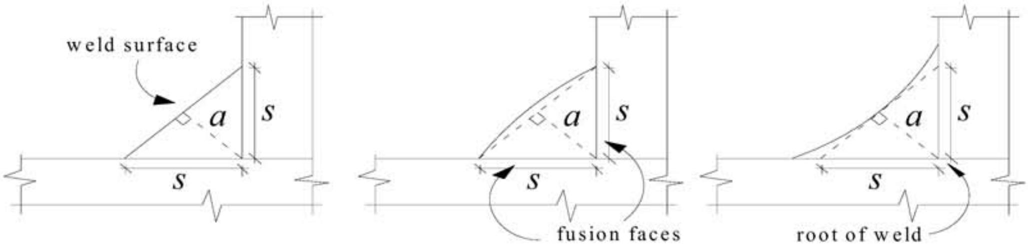
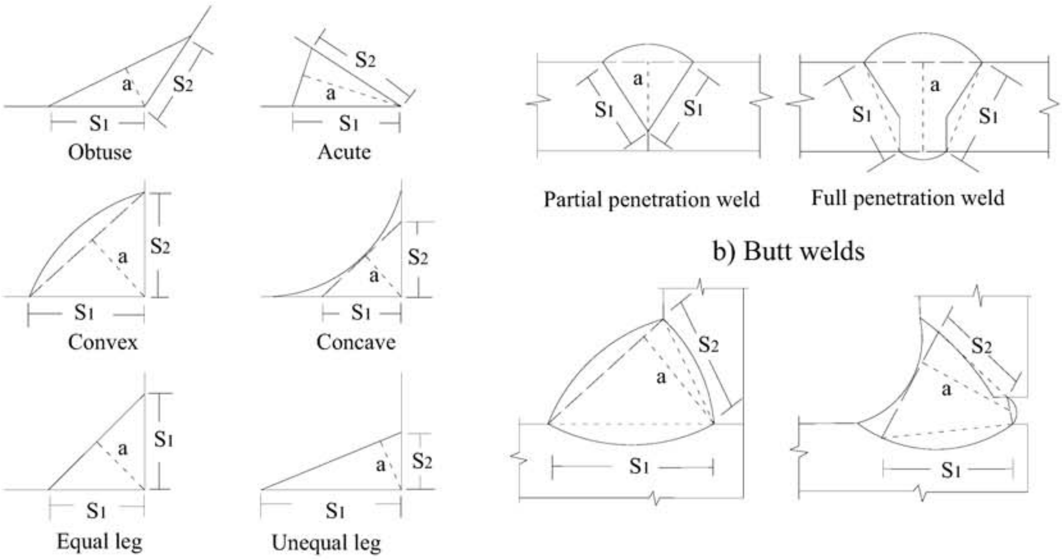


Figure 7 Equal leg length of typical fillet welds

For more complex connections, the throat size can be determined from engineering assessment and below are some of the examples for location of the throat size. In Figure 8 (a), the throat thickness a is taken as the shortest distance from the root of weld to the fusion surface and s_1 and s_2 denote the leg lengths on both sides parallel to the parent metals. The throat thickness a for a butt weld can be taken as perpendicular distance from root of weld as indicated in Figure 8 (b). In the cases of deep fillet welds, throat thickness a is also taken shortest distance from root of weld as shown in Figure 8 (c). For design calculation and drawing preparation, leg length with equal magnitude on two sides is normally specified.



a) Fillet welds c) Deep fillet welds

Figure 8 Definitions of sizes of fillet weld and butt weld

The strength p_w of a fillet weld depends not only on the strength of parent metal, but also on the material used in the welding electrode.

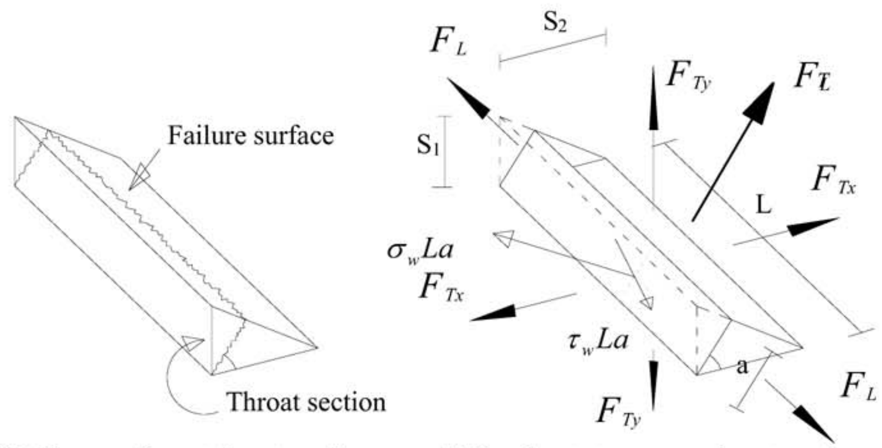
When two different grades of parent materials are joined by fillet welds, the lower grade should be considered in design. For example the design strength p_w of fillet welds for standard steel grade and common electrode type are tabulated in Table A below.

Table A Design strength of fillet welds p_w to BS-EN standards

Steel grade	Electrode classification		
	35	42	50
	Strength (N/mm^2)	Strength (N/mm^2)	Strength (N/mm^2)
S275	220	220	220
S355	220	250	250
S460	220	250	280

In addition to the design calculation, a fillet weld is required to be returned around corners for at least twice of the leg length and the lap length in a lap joint should not be less than 4 times the thickness of the thinner plates.

In general, the failure surface of weld is approximately at the throat section under longitudinal and transverse forces as shown in Figure 9 (a). The forces on a fillet weld of length L and throat thickness a are illustrated in Figure 9 (b).



a) Failure surface at throat section b) Resultant stresses on throat section

Figure 9 Resultant stresses acting on the fillet weld of throat section

The force on a particular weld due to moment and shear in a connection can be resolved into the directions parallel and longitudinal to the weld and then checked against the design capacities of the weld in these two directions as follows. The longitudinal design capacity per unit length of weld P_L is given by the following expression.

$$P_L = p_w a$$

in which p_w is the design strength of weld obtained from Table A above and a is the throat size of the weld.

The capacity per unit length of the weld in the transverse direction P_T is given by,

$$P_T = KP_L$$

in which P_L is design capacity per unit length of weld and K is a coefficient given by,

$$K = 1.25\sqrt{\frac{1.5}{1+\cos^2\theta}}$$

in which θ is the angle between the resultant and the line bisecting the area of the weld.

The external force acting on the weld can be resolved into the components in the longitudinal and transverse directions of the weld as shown in Figure 10 (a) & (b) and determined as F_L and F_T which is equal to $F_T = \sqrt{F_{Tx}^2 + F_{Ty}^2}$ as shown in Figure 10 (c). The structural adequacy of the weld can be checked by the conditions as,

$$P_L \geq F_L$$

$$P_T \geq F_L$$

$$\left(\frac{F_L}{P_L}\right)^2 + \left(\frac{F_T}{P_T}\right)^2 \leq 1$$

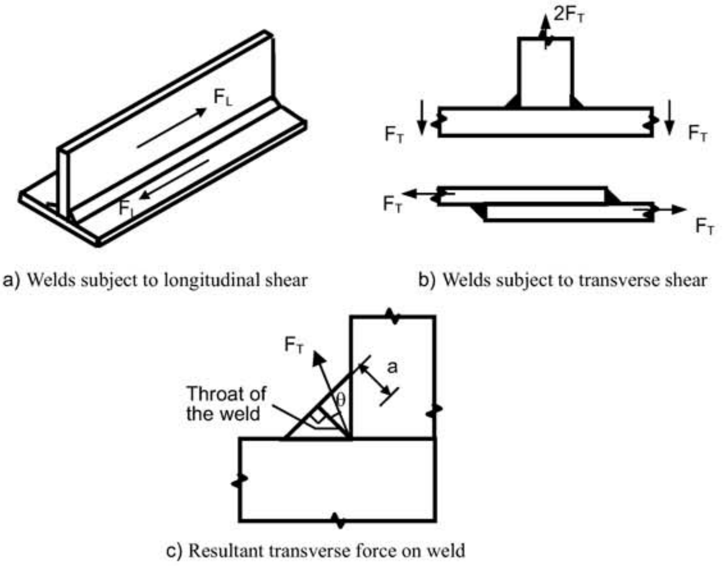
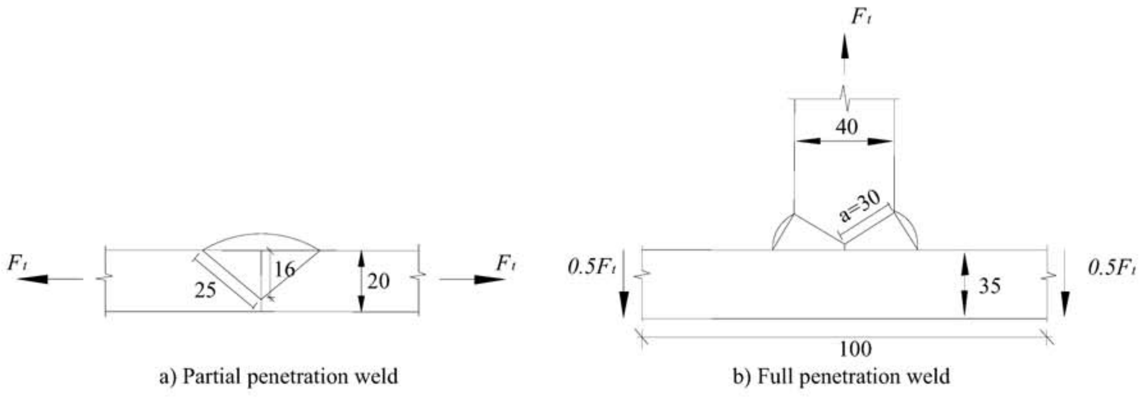


Figure 10 Directional approach for capacity of fillet weld

g) Worked example on simple welded connection

The connection is formed by joining two plates together by butt weld as shown. The parent metal is in grade S460. The welds are used to transfer tension only, which are 200kN and 1520kN for cases (a) and (b) respectively. The sizes of butt weld are also given in the Figure. The electrode of weld is in both cases E50.

Length of connections for both cases in longitudinal direction is 300mm.



<u>Solution</u>

a) Partial penetration weld

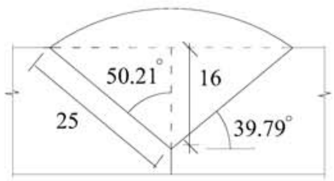
TENSION CAPACITY OF WELD:
Effective throat thickness, $a = 16mm < 0.7s = 0.7 \times 25 = 17.5mm$

Design strength of weld, $p_w = 280N/mm^2$
Capacity of fillet weld, $P_L = p_w a = 280 \times 16 = 4.48kN/mm$ (transverse to the weld)

Total resistance of this connection, $P_T = 4.48 \times 300 = 1{,}344kN > F_t = 200kN$

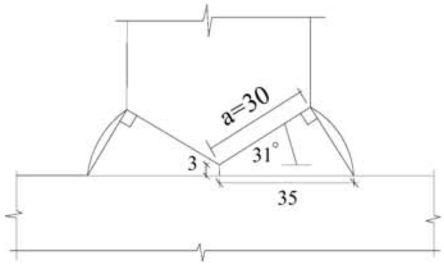
b) Full penetration weld

TENSION CAPACITY OF VERTICAL PLATE ELEMENT:
Effective throat thickness, $a = 30mm > 0.7s = 0.7 \times 30 = 21mm$ (it is butt weld)



The strength of butt weld is as strong as the parent metal, so it is not necessary to check the capacity of connection. However, the tension capacity of vertical plate element and bending capacity of horizontal plate element should be checked.

Design strength of parent metal, $p_w = 440N/mm^2$ ($T \leq 40mm$)



Effective area of vertical plate element, $A_e = 40 \times 300 = 12000mm^2$
Tension capacity of vertical plate element, $P_t = p_y A_e = 440 \times 12000$
$= 5280kN > F_t = 1520kN$

When the effective length b_e of weld is less than 40mm, the weld length is so small that it cannot be assumed to take any load. Also, the section properties of welded connection should be based on the effective section obtained from the effective length section.

5. RECOMMENDATIONS ON ADDITIONAL MEASURES ON DESIGN OF WELD

In addition to the above static strength calculations, attention must be paid to prevent the occurrence of the following possible welded defects: -

I. Lamellar tearing

II. Preheat treatment to mitigate hydrogen embrittlement

I. Lamellar tearing

Steel plate used to manufactured by rolling from large ingot to thin plate. In modern production, this has often been replaced by the continuous casting process (con-cast). In the course of either process, if inclusions of oxide or sulphide are present, they can be rolled to form planes of weakness. In welding an attachment to the surface of the plate, contraction of weld sets up through thickness strains and if the plate material is susceptible, it may fail by lamellar tearing.

One way to avoid lamellar tearing is to design joints such as the tensile stress does not act on the through thickness direction. As a general guide, T-butt or cruxiform welds should always be designed with thinner plate set on to the surface of the thicker plate. A certain extent of mitigation may be achieved by removing a surface layer of plate to depth of say 5mm and fill with a layer of weld metal (buttering) or by using as low strength weld metal as available and permissible.

II. Preheat treatment to mitigate hydrogen embrittlement

During welding, hydrogen in atomic form can be absorbed by the molten weld metal from the arc atmosphere. On subsequent cooling, the solubility of hydrogen decreases as much of the hydrogen atoms escaped by diffusion into the air but some may be trapped in the heat affected zone (HAZ) and the parent metal. Control of the composition of the coating in electrodes and baking of electrodes before welding to drive off moisture are essential to keep the potential hydrogen level low. [This hydrogen atoms can stray in the lattice structure and weaken the interatomic strength among the atoms].

To prevent formation of hydrogen cracks in HAZ regions and welds and to mitigate the distortion effects, preheat treatment is applied to reduce cooling rate and hence to minimize the degree of hardening close to fusion line. The object of controlling the preheat and interpass temperatures for most Carbon-Manganese and ferritic alloy steels (body-centre-cubic lattice structure) is to minimize the risk of hydrogen cracking.

The welding procedure should conform to BS EN 1011, which defines procedures to minimize the risk of hydrogen cracking in weld metal and/or HAZ. The first requirement is to establish a correction level of preheat, which depends on four variables :-

a. CE (Carbon Equivalent) of plate material
b. combined joint thickness
c. amount of hydrogen introduced into the weld
d. heat input value

Basic low hydrogen coating electrodes should be used where hydrogen cracking is considered as hazardous and where weld metal is required to have good notch-ductility.

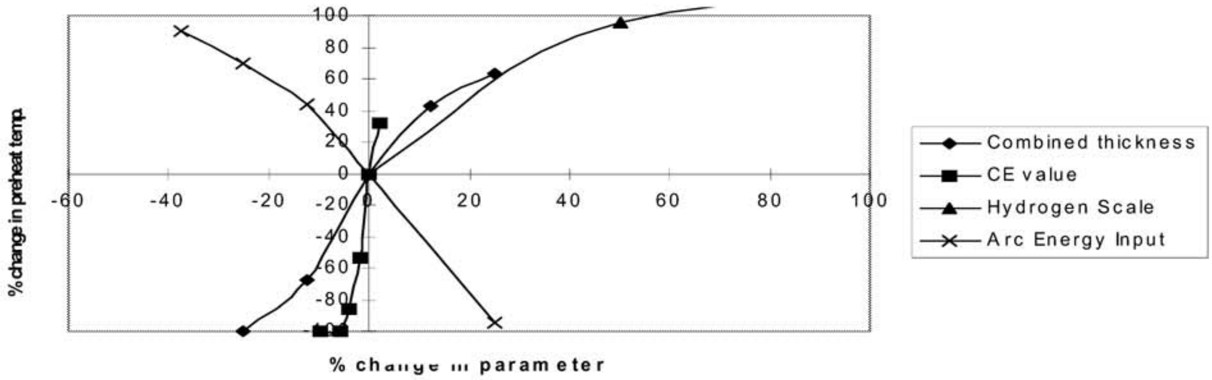


Figure 11 Combined thickness = 80mm CE value = 0.51
Hydrogen Scale = D Arc Energy Input = 2.0KJ/mm
Preheat temperature = 62.5°C

BS EN 1011 would suggest to use low hydrogen electrode for grade S355 steel and above. Preheat is not required as a general rule but for thicker material, it is desirable to limit CE value so as to minimize the level of preheat.

Figure 11 describes the sensitivity of the four key parameters on the preheat requirement at the welded connection. It is obvious to conclude that the steeper the gradient of the parameter, the higher is the degree of sensitivity over the preheat treatment. In the above case, the order of high sensitivity is Carbon Equivalent value (CEV), followed by the Combined Thickness, Hydrogen potential of the electrode and heat energy input respectively.

5. RECOMMENDATIONS ON WORKING DETAILS DURING FABRICATION OF WELD

I. End returns

The following end return details are good welding practice so as to alleviate the stress concentration at the ends of the weld.

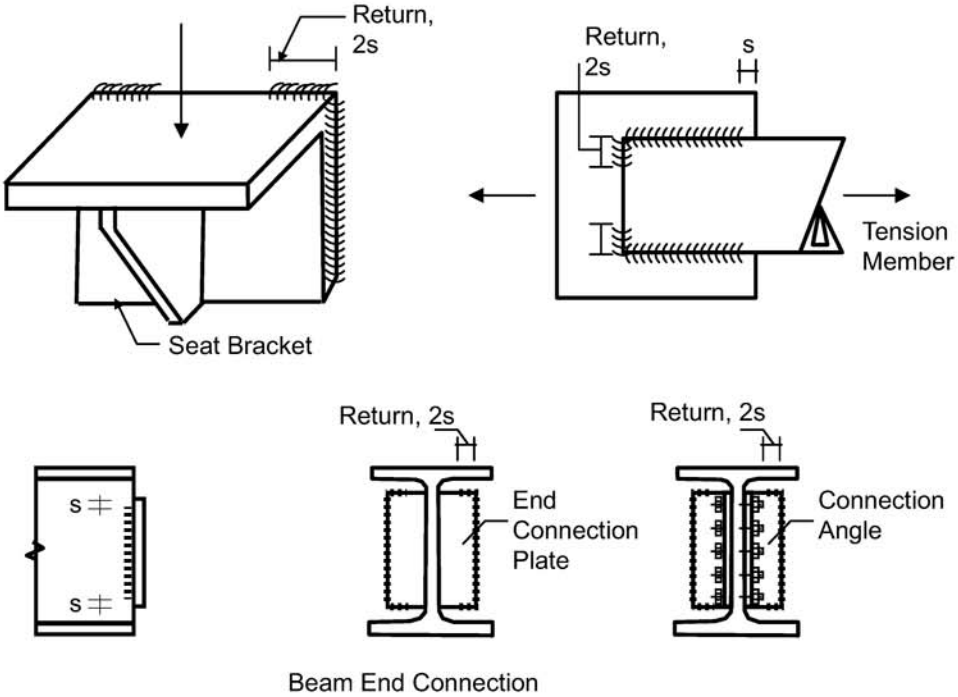
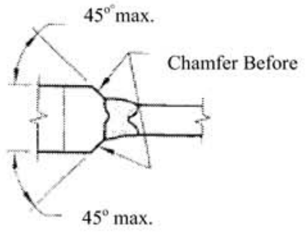


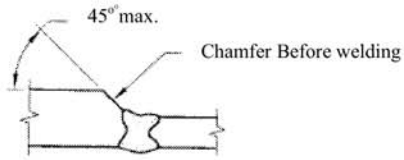
Figure 12 End return of welds

II. Transitional chamfer of joining plates of different thicknesses



Centre-line alignment

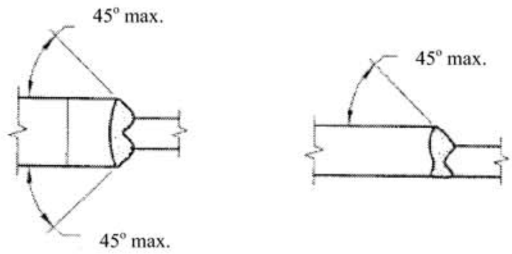
(particular applicable to web plates)



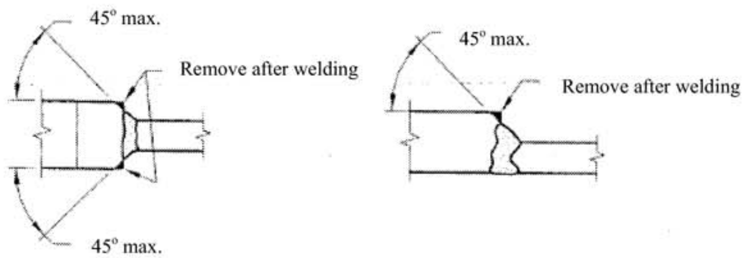
Offset alignment

(Particular applicable to flange plates)

Transition by chamfering thicker part

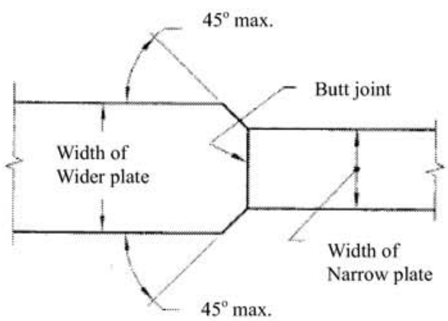


Transition by sloping weld surface



Transition by sloping weld surface and chamfering

Transition of butt joints in parts of unequal thickness



Transition of butt joints in parts of unequal width,
transition by chamfering wider part

Figure 13 Transition of thickness or widths for butt weld subject to tension

III. Distortion

In fusion welding, the metal immediately surrounding a weld, being restrained by cooler areas, is first subject to compressive stress at elevated temperature. If the metal yields, it will be in a state of residual tensile stress on cooling. Weld metal starts to contract as it solidifies. In thin sections, this contraction results in distortion and buckling. Therefore, the welded joint should be so designed that the amount of weld metal is minimized and hence the degree of contraction is reduced.

IV. Relief of residual tensile stress by post-weld heat treatment

Residual tensile stress, balanced by compressive stress, may cause cracking in welds that have suffered embrittlement or in plate where restraint is high and where an initiating crack is present. In ductile metals not subject to stress corrosion cracking, residual stresses have little or no effect on behaviour of welded joint.

Post-weld heat treatment is a means to relieve residual tensile stress if this is necessary. Since contraction stresses can build up to level that can initiate a brittle crack; it is also a means to release Hydrogen trapped in the grain boundaries.

Normalizing, viz. heat treatment in austenitic temperature ranging up to 850°C followed by cooling in still air down to room temperature, is a form of post-weld heat treatment, which practically removes residual tensile stresses and restores the microstructure and properties.

V. Slag inclusions

The character of weld metal is governed by the chemical action of flux coated around the electrode, the measurement of which is based on Basicity Index. These flux would directly affect the character of slag, which would affect the nature of weld metal. This commonly used index for Basicity is adopted by the International Institute of Welding (IIW) and is as follows :-

$$BI = \frac{CaO + CaF_2 + MgO + K_2O + Na_2O + 1/2(MnO + FeO)}{SiO_2 + 1/2(Al_2O_3 + TiO_2 + ZrO_2)}$$

For flux of Basicity Index < = 1, it is acidic.
For flux of Basicity Index between 1.0 and 1.5, it is neutral.
For flux of Basicity Index between 1.5 and 2.5, it is semi basic.
For flux of Basicity Index > 2.5, it is basic.

An acid compound of slag is less stable than basic component of slag. Therefore, acid slags produce a weld metal of higher oxygen content than basic slags. Higher basicity tends to reduce Sulphur content of weld and is normally beneficial. Where the Basicity Index (BI) increases, Silicon decreases and Manganese increases.

The notch ductility is a toughness or impact property of material and is usually described as the impact energy versus temperature. The notch ductility of weld metal is improved by lower oxygen, lower Silicon and higher Manganese, i.e. basic flux are to be used. Hence, basic coated electrodes can produce carbon steel deposits that have higher impact properties than those deposited with cellulosic or rutile coated electrodes.

6. HIGHLIGHTS ON SOME BAD WELDING PRACTICE

Single-sided butt welds should not be used where the weld is put under bending, especially when it puts the root in tension. See figure 14 below.

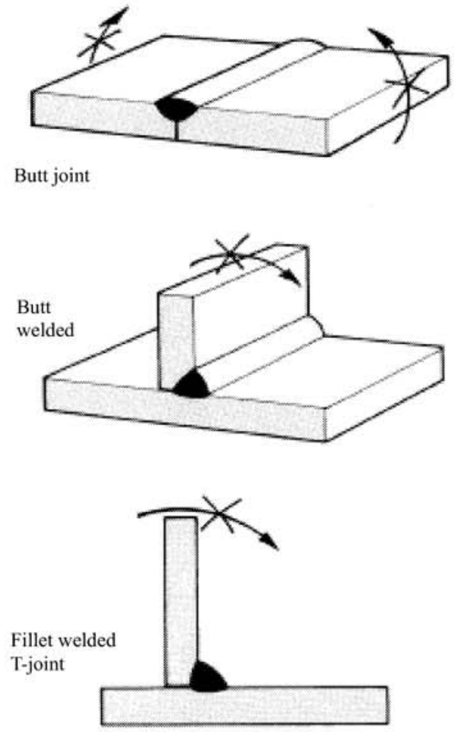


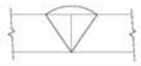
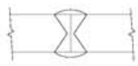
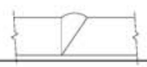
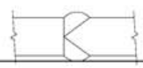
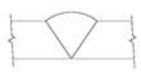
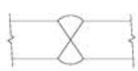
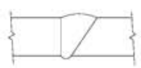
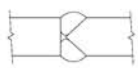
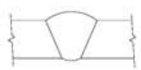
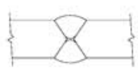
Figure 14 Bad welding practice

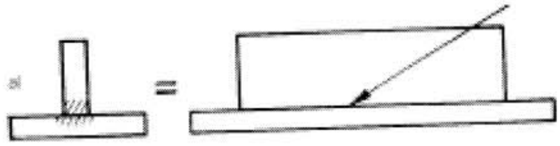
7. CONVENTIONAL WELD SYMBOLS

It is always assumed by the designers that fabricators automatically understand what type or size of weld is to be used. However, this is not the case at all. Welds should be shown on drawing as pictures of what they look like. The welding symbols are given in Appendix C of the Code. Here are some simple examples showing the common welding symbols.

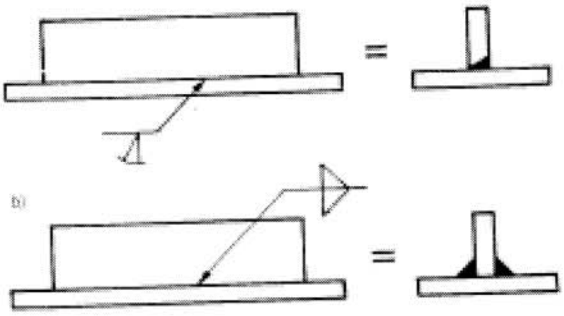
In shop drawings and fabrication plan as per figure 15, the welds are shown on the type, size, length and locations on the connected parts. This information is indicated in form of symbols. The commonly used symbols are indicated in the Table B below.

Table B Common arrangements for welding

Weld type		Single	Double
Groove	Square		
	Bevel		
	Vee		
	J		
	U		



Showing position of Weld



Showing weld type

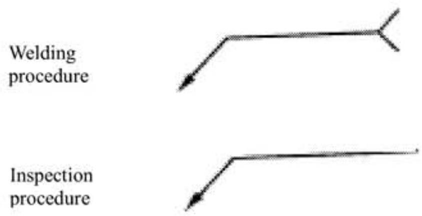


Figure 15 Conventional welding symbols
giving welding and inspection procedure

8. CURRENT MATERIALS AND WORKMANSHIP STANDARDS

Welding is a well established skill in which the use of welding consumables, welding procedures, destructive tests and non-destructive test of welds are well governed by competent standards as given in Appendix I. Obsolete standards are given against the current standards, to which practitioners are recommended to refer. It is encouraged that the practitioners should adhere to the use of the current standards, albeit the stakeholders in local construction industry, such as the steel suppliers and HOKLAS accredited testing laboratories, may need to take some time to adapt to the transitional change.

9. OVERVIEW OF NATIONAL CODES ON DESIGN OF WELDS

An overview of the national codes enlisted below was conducted on the design of welds.

a) American Code : Load and Resistance Factor Design (LRFD): 1999
b) Australian Code – AS/NZS 1554-1:2000
c) British Code – BS 449-2: 1969 (now obsolete)
d) British Code - BS 5950: Part 1: 1990
e) British Code - BS 5950: Part 1: 2000
f) Chinese Code - GB 50017-2003
g) European Code – EN 1993-1-8: 2005
h) Hong Kong - Code of Practice for the Structural Use of Steel 1987
i) Hong Kong - Code of Practice for the Structural Use of Steel 2005
j) The Building Standard Law of Japan: 1990

The following aspects are studied and a comparison was summarized in Appendix II: -

- Design approach
- Fillet weld strength
- Intermittent welds
- Weld leg length

10. CONCLUSIONS

The paper endeavours to give a general review on the design and fabrication of welded connections and some suggestions are recommended so as to sustain the weld quality.

ACKNOWLEDGMENT

This paper is indebted to the full support of Professor F M Burdekin and Professor S L Chan for their expert advice. I would take this opportunity to express my gratitude to the Buildings Department in permitting me to publish the paper.

REFERENCES

1. American Code : LRFD: 1999 - Load and Resistance Factor Design Specification.

2. Australian Code – AS/NZS 1554-1: 2000 – Structural steel welding. Part 1: Welding of steel structures.

3. British Code – BS 449-2: 1969 – Specification for the use of structural steel in building. Part 2. Metric units.

4. British Code - BS 5950: Part 1: 1990 – Structural use of steelwork in building. Part 1: Code of practice for design in simple and continuous construction: hot rolled sections.

5. British Code - BS 5950: Part 1: 2000 – Structural use of steelwork in building. Part 1: Code of practice for design – Rolled and welded sections.

6. Chinese Code - GB 50017-2003 – Code for design of steel structures.

7. European Code – EN 1993-1-8: 2005 – Eurocode 3: Design of steel structures – Part 1-1: General rules and rules for buildings.

8. Hong Kong - Code of Practice for the Structural Use of Steel 1987 .

9. Hong Kong - Code of Practice for the Structural Use of Steel 2005.

10. The Building Standard Law of Japan: 1990.

Appendix I

Comparison of BS and BS EN for Welding consumables

Title of BS	BS	BS EN	Title of BS EN
Covered carbon and carbon manganese steel electrodes for manual metal-arc welding	BS 639:1986 (Superseded, Withdrawn)	BS EN 499: 1995 (Superseded, Withdrawn) by BS EN ISO 2560: 2005 (Current)	Welding consumables- Covered electrodes for manual metal arc welding of non-alloy and fine grain steels – Classification Welding consumables- Covered electrodes for manual metal arc welding of non-alloy and fine grain steels - Classification
Filler rods and wires for gas-shielded arc welding. Part 1: Ferritic steels	BS 2901-1: 1983 (Superseded, Withdrawn)	BS EN 440: 1995 (Current)	Welding consumables- Wire electrodes and deposits for gas shielded metal arc welding of non-alloy and fine grain steels - Classification
Specification for electrode wires and fluxes for the submerged arc welding of carbon steel and medium-tensile steel	BS 4165: 1984 (Superseded, Withdrawn)	BS EN 756: 2004 (Current)	Welding consumables- Solid wires, solid wire-flux and tubular cored electrode-flux combinations for submerged arc welding of non-alloy and fine grain steels – Classification
Specification for carbon and carbon-manganese steel tubular cored welding electrodes	BS 7084: 1989 (Revised, Withdrawn)	BS EN 758: 1997 (Current)	Welding consumables- Tubular cored electrodes for metal arc welding with and without a gas shield of non-alloy and fine grain steels – Classification

Comparison of BS and BS EN for Welding Standard, Procedure, Coordination and Welder

Title of BS	BS	BS EN	Title of BS EN
Arc welding of carbon and carbon manganese steels	BS 5135:1984 (Superseded, Withdrawn)		Welding – Recommendations for welding of metallic materials
		BS EN 1011-1:1998 (Current)	Part 1: General guidance for arc welding
		BS EN 1011-2:2001 (Current)	Part 2: Arc welding of ferritic steels
Specification for approval testing of welding procedures. Part 1: Fusion welding of steel.	BS 4870-1:1981 (Superseded, Withdrawn)	BS EN ISO 15614-1: 2004 (Current)	Specification and qualification of welding procedures for metallic materials. Welding procedure test. Part 1: Arc and gas welding of steels and arc welding of nickel and nickel alloys
Part 3: Arc welding of tube to tube-plate joints in metallic materials	BS 4870-3:1985 (Superseded, Withdrawn)	BS EN ISO 15614-8:2002 (Current)	Specification and qualification of welding procedures for metallic materials – welding procedure test Part 8: Welding of tubes to tube-plate joints
Part 4: Specification for automatic fusion welding of metallic materials, including welding operator approval	BS 4870-4:1988 (Superseded, Withdrawn)	BS EN 1418: 1998 (Current)	Welding personnel. Approval testing of welding operators for fusion welding and resistance weld setters for fully mechanized and automatic welding of metallic materials.
	-	BS EN 719: 1994 (Current)	Welding coordination – Tasks and responsibilities

Title of BS	BS	BS EN	Title of BS EN
	-	BS EN 729-2: 1995 (Superseded, Withdrawn) by BS EN ISO 3834-2:2005 (Current)	Quality requirements for welding. Fusion welding of metallic materials. Comprehensive quality requirements. Quality requirements for fusion welding of metallic materials. Part 2: Comprehensive quality requirements
	-	BS EN 729-3: 1995 (Superseded, Withdrawn) by BS EN ISO 3834-3:2005 (Current)	Quality requirements for welding. Fusion welding of metallic materials. Part 3: Standard quality requirements. Quality requirements for fusion welding of metallic materials. Part 3: Standard quality requirements
	-	BS EN 729-4:1995 (Superseded, Withdrawn) by BS EN ISO 3834-4:2005 (Current)	Quality requirements for welding. Fusion welding of metallic materials. Part 4: Elementary quality requirements. Quality requirements for fusion welding of metallic materials. Part 4: Elementary quality requirements
Specification for approval testing of welders to approved welding procedures. Fusion welding of steel	BS 4871-1:1982 (Superseded, Withdrawn)	BS EN 287-1:2004 (Current)	Qualification test of welders. Fusion welding. Part 1: Steels

Title of BS	BS	BS EN	Title of BS EN
Specification for approval testing of welders to approved welding procedures. Part 3: Arc welding of tube to tube-plate joints in metallic materials	BS 4871-3:1985 (Current)		
Specification for approval testing of welders when welding procedure approval is not required Part 1: Fusion welding of steel	BS 4872-1: 1982 (Current)		

Comparison of BS and BS EN for DT of welds

Title of BS	BS	BS EN	Title of BS EN
Methods of destructive testing, fusion welded joints and weld metal in steel	BS 709:1983 (Superseded, Withdrawn)	BS EN 875: 1995 (Current)	Destructive tests on welds in metallic materials. Impact tests. Test specimen location, notch orientation and examination.
		BS EN 876: 1995 (Current)	Destructive tests on welds in metallic materials. Longitudinal tensile test on weld metal in fusion welded joints.
		BS EN 895: 1995 (Current)	Destructive tests on welds in metallic materials. Transverse tensile test.
		BS EN 910: 1996 (Current)	Destructive tests on welds in metallic materials. Bend tests.
		BS EN 1043-1: 1996 (Current)	Destructive tests on welds in metallic materials. Hardness testing. Part 1: Hardness test on arc welded joints.

		BS EN 1320: 1997 (Current)	Destructive test on welds in metallic materials. Fracture tests.
		BS EN 1321: 1997 (Current)	Destructive test on welds in metallic materials. Macroscopic and microscopic examination of welds

Comparison of BS and BS EN for NDT of welds

Title of BS	BS	BS EN	Title of BS EN
Code of practice. Visual inspection of fusion welded joints.	BS 5289:1976 (Superseded, Withdrawn)	BS EN 970:1997 (Current)	Non-destructive examination of fusion welds – Visual examination
Method for penetrant flaw detection.	BS 6443:1984 (Superseded, Withdrawn)	BS EN 571-1:1997 (Current)	Non-destructive testing – Penetrant testing Part 1: General principles
Method for magnetic particle flaw detection	BS 6072:1981 (Current, Obsolescent, Superseded)	BS EN 1290:1998 (Current) BS EN ISO 9934-1:2001 (Current)	Non-destructive examination of welds – Magnetic particle examination of welds Non-destructive testing. Magnetic particle testing. Part 1: General principles.
Method for ultrasonic examination of welds Part 1: Methods for manual examination of fusion welds in ferritic steels	BS 3923-1:1986 (Superseded, Withdrawn)	BS EN 1714:1998 (Current)	Non-destructive testing of welded joints – Ultrasonic testing of welded joints
Radiographic examination of fusion welded butt joints in steel. Part 1: Methods for steel 2 mm up to and including 50 mm thick Part 2: Methods for steel over 50 mm up to	BS 2600-1 & 2:1983 (Superseded, Withdrawn)	BS EN 1435:1997 (Current)	Non-destructive examination of welds. Radiographic examination of welded joints.

and including 200 mm thick			

Overview of various national codes on design of welds

Appendix II

Code	Design approach	Fillet weld strength	Intermittent Weld	Weld leg length (mm)
LRFD: 1999 (USA)	Limit state approach	Using resistance factor ϕ multiplied to specified strength of weld to derive design strength	FW Weld length > 4 x weld size, 38 mm min	FW <6 3 6-13 5 13-19 6 >19 8
AS/NZS 1554.1:2000 (Australia)	Limit state approach	No particular mention	FW Weld length>4t, 40mm min	FW <3 2t/3 3-7 3 7-10 4 10-15 5 >15 6
BS 449-2: 1969 (UK) (obsolete)	Permissible stress approach	115 MPa for gr.43 160 MPa for gr.50 195 MPa for gr.55	BW Weld length<12t Clear dist btn weld<12t FW Clear dist btn weld <16t for compression <24t for tension in no case> 300mm	No particular mention
BS 5950: Part 1: 1990 (UK)	Limit state approach	Electrode class 35 42 50 design strength 40/43 215 215 50 215 255 55 255 275	FW Clear dist btn weld <16t for compression <24t for tension in no case> 300mm	Min. throat thk for partial penetration butt weld = $2\sqrt{t}$
BS 5950: Part 1: 2000 (UK)	Limit state approach	Electrode class 35 42 50 design strength S275 220 220 220 S355 220 250 250 S460 220 250 280	FW Clear dist btn weld <16t for compression <24t for tension in no case> 300mm	Min. throat thk for partial penetration butt weld = $2\sqrt{t}$

Code	Design approach	Fillet weld strength	Intermittent Fillet Weld	Weld leg length
GB 50017-2003 (China)	Limit state approach	Electrode strength class Q235 E43 160 Q345 E50 200 Q390/420 E55 220	FW Weld length>10t, 50 min Clear dist btn weld <15t for compression <30t for tension	FW Leg length=1.5√t t<6, leg length <t t>6, leg length <t-(1~2)
EN 1993-1-8:2005 (Europe)	Limit state approach	Vector stress < Ult strength/ (βw x γ_M) $\sigma\perp$ < 0.9 Ult strength/γ_M where βw is correlation factor	FW Distance btn eff. Length < 12t for comp / shear < 16t for tension in no case > 200 mm Design resistance enhanced by (e+l)/l, where e is clear distance btn weld and l is weld length	No particular mention
CoP Steel 1987 (HK)	Permissible stress approach	Strength Grade 115 MPa for gr.250 160 MPa for gr.350 195 MPa for gr.450	FW: Distance btn eff. Length <16t for compression <24t for tension in no case> 300mm	No particular mention
CoP Steel 2005 (HK)	Limit state approach	Electrode class 35 42 50 design strength S275 220 220 220 S355 220 250 250 S460 250 280	FW: Distance btn eff. Length <16t for compression <24t for tension in no case> 300mm	FW <6, 3 7-13 5 14-19 6 >19 8
The Building Law of Japan: 1990 (Japan)	Permissible stress approach	F/(1.5x√3) For temp works, F/√3	No particular mention	No particular mention

Note:

FW : Fillet weld BW : Butt weld t : thickness of thinner attachment plate weld size (LRFD) = Leg length, s

F : Standard strength of weld material

COMPOSITE JOINTS WITH STEEL BEAMS AND PRECAST HOLLOWCORE SLABS

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ABSTRACT

Composite design incorporating steel beams and precast hollowcore slabs is a recently developed composite floor system for building structures. As the construction industry demands for rapid construction with reduction in cost and environmental impacts, this form of composite floor system, which does not require major onsite concreting, has become very popular among the designers and engineers in the UK. This form of composite construction is so far limited to simple beam-column connections. The concept of semi-rigid composite connection has been widely research in the past; however, most of the researches are limited to composite joints with metal deck flooring and solid concrete slabs. Semi-rigid composite joint incorporating steel beams and precast hollowcore slabs is developed which can provide sufficient moment and rotation capacity in composite beams design. In addition to full-scale joint tests, a finite element model to simulate the structural behaviour of the composite connections was developed and used to study the behaviour of a wide range of composite joints to gain a better understanding of the structural behaviour especially the moment-rotation characteristic of the connections. Parametric studies were carried out to investigate the structural behaviour with variations parameters. Through the parametric study, an analytical model for the semi-rigid composite joints is proposed and is verified with both the experimental data and finite element model and good agreement is obtained.

Keywords: Semi-rigid, composite, joints, precast, hollowcore, steel, connections, beam-column

1. INTRODUCTION

In the area of composite research, extensive works have been focused on semi-rigid connections design since it was first proposed by Barnard (1) in the 70s. It showed these form of connections when used in design will lead to reduction in beam sizes, which in turn will reduce the beam depth, the overall building height and cladding cost, etc. Moment rotation characteristic of the semi-rigid composite connections was first investigated by Johnson and Hope-Gill (2) in 1972; they found that neither simple nor rigid beam-column connections are ideal. Simple joints are too unpredictable while rigid joints are often too stiff in relation to their strength and are expensive; therefore, the semi-rigid joint with a large rotation capacity and a predictable flexural strength that does not require site welding or accurate fitting is needed. Numerous researches have been carried out on semi-rigid composite connections (3,

4) mainly with solid R.C. slabs and profiled metal deck floors. Semi-rigid composite joint incorporating steel beams and precast hollowcore slabs is a newly developed system for the building construction. The most important properties of this type of connections are moment capacity, rotational stiffness and rotational capacity. These parameters can be defined using the moment-rotation response (M-θ). The best way to capture the M-θ response is by conducting full scale tests. Extensive full scale tests have been done over the last three decades, but fewer have been done with the precast hollowcore slabs. The first commercial testing in this area was carried out at Salford University and reported by Hamilton (5). The mode of failure was due to shearing off of the headed studs. Research on shear connector strength in precast solid concrete planks was carried out by Moy and Tayler (6). Twenty-seven push-off tests were carried out and the results showed that a reduction in strength as the gap of in-situ concrete decreased. Tests on composite beams with precast solid planks were also conducted by Jolly (7) at the Southampton University. A 16m span composite beam with 110mm deep precast concrete planks was tested by Shim et al(8) studied the behaviour of headed shear studs in precast post-tensioned bridge decks at the Seoul National University. Push-out tests were carried out to determine the structural behaviour of the shear connection in precast deck. Full size push-off tests with precast hollowcore slabs were first performed by Lam et al (9) in 1998. Three full scale simply supported composite beams with variable parameters were also carried out to study the flexure behaviour and was compared with non-composite bare steel beams (10). Due to the limitation of the test results, non-linear finite elements method is an attractive tool for modelling the connections. The use of finite element modelling could explore large number of variables and potential failure modes, which could complement the experimental studies. Lam et al (11) were the first to simulate the behaviour of composite girders with precast hollowcore slabs; a 2-D finite element model was built using ABAQUS (12). A 3-D FE model of the headed studs in steel-precast composite beams was built by El-Lobody and Lam (13) using ABAQUS to model the behaviour of the headed stud in precast hollowcore slabs; elastic-plastic material was used for the simulation. The model was validated against the test results and good agreement is obtained.

From the available literatures, it is noted that although there are some research works toward the finite element model of composite construction, most of the works were towards the simulation of the composite beams, fewer works have been done to model the composite joints, especially using a 3-D finite element method. However, the use of the nonlinear moment-rotation curve from the test or FE model is far too complex and impractical for designers. To solve this problem, a simple but accurate analytical method to calculate the moment and rotation capacities for this form of composite joint is badly needed. In this paper, an analytical method for calculating the moment and rotation capacity is presented and comparison with the full-scale tests result is made to validate its accuracy.

2. FULL SCALE TESTS

Full-scale joint tests with flush endplate composite connection and precast hollowcore slabs were conducted by Fu and Lam (14). The main variables investigated were stud spacing, degree of the shear connections, amount of the longitudinal reinforcement and slab thickness. All specimens were of cruciform arrangement as shown in Figure 1 to replicate the internal beam-column joints in a semi-rigid composite frame. The specimen was assembled from two 3300 mm long 457×191×89UB grade S275 universal beams and one 254×254×167UC grade S275 universal column to form the cruciform arrangement. The beams were connected to the column flanges using 10mm thick flush end plates with two rows of M20 Grade 8.8 bolts. A single row of 19mm diameter headed shear studs were pre-welded to the top flange of the steel beams. The steel connection shown in Figure 2 is a typical connection currently used in UK practice for simple joint.

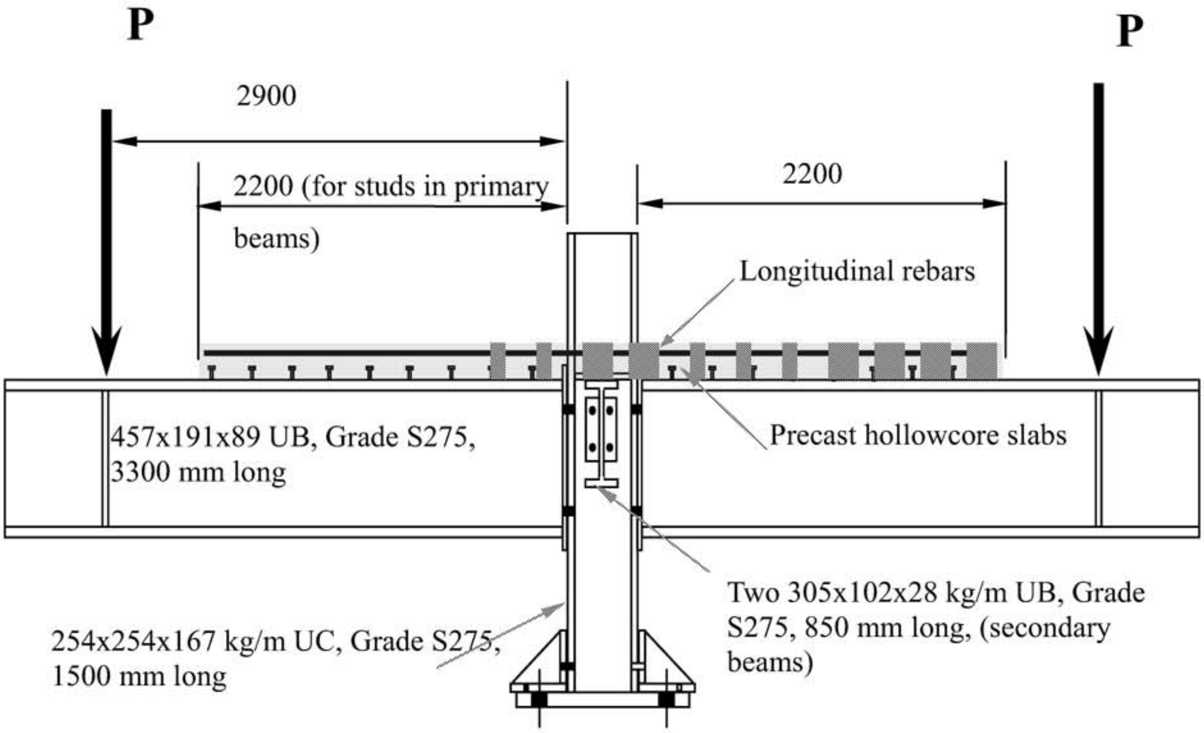

Figure 1 General arrangement of test set-up

Results of all the composite joint tests are shown in Table 1 and Figure 3. All tests except Test CJ3 failed in a ductile manner with beam rotation well in excess of 30 mRad and obtained a moment capacity above 0.3 Mp of the composite beams, it can be concluded that these types of joints can provide sufficient moment and rotation capacity for plastic design. Tests CJ1, CJ2, CJ6, CJ7, and CJ8 were failed due to the fracture of longitudinal reinforcement while Tests CJ3, CJ4 and CJ5 failed by fracture of the shear connectors. No yielding or buckling to the column was observed. For all the tests conducted, no bond failure between the in-situ and the precast concrete was observed, therefore it can be concluded that the in-situ and the precast hollowcore slabs were acting compositely throughout.

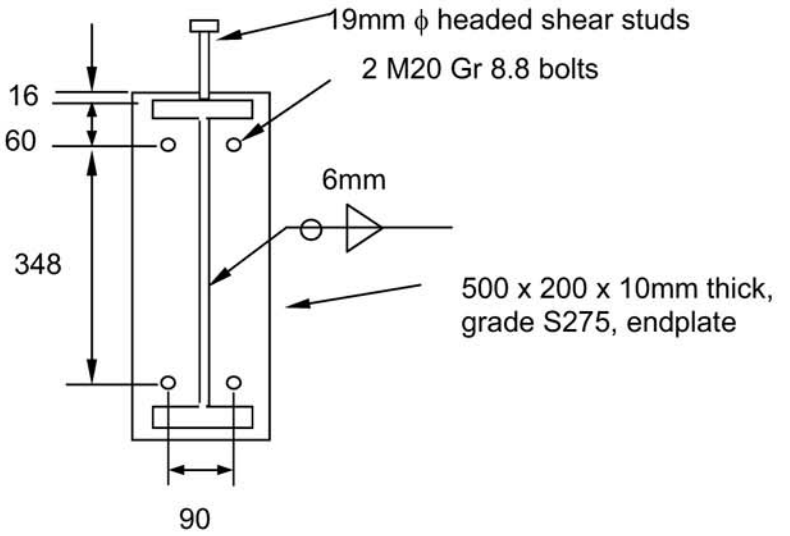

Figure 2 End plate connection

TABLE 1 TESTS RESULT

Reference	**CJ1**	**CJ2**	**CJ3**	**CJ4**	**CJ5**	**CJ6**	**CJ7**	**CJ8**
Moment capacity (kNm)	370	363	250	368	363	425	274	439
Rotation capacity (mRad)	35.4	33.5	6.1	37.4	31.7	46.8	30	42.3
Long. reinf. – yield (kN)	326	326	326	326	326	424	212	424
Long. reinf. – Ultimate (kN)	387	387	387	387	387	486	243	486
Shear connector capacity (kN)	896	512	256	384	384	512	256	512
Degree of shear connection (%) [1]	>100	>100	78.5	>100	>100	>100	>100	>100
Degree of shear connection (%) [2]	>100	>100	66	98	98	>100	>100	>100
Max. strain in long. reinf. (με)	26,000	23,000	2,031	16,000	13,706	26,000	23,000	23,000
Maximum end slip (mm)	0.34	0.8	5.8	3.5	3.5	0.84	0.4	1.6
Failure mode	RF	RF	CF & SF	CF	CF	RF	RF	RF

RF – reinforcement fracture; CF – connector fracture; SF – slab shear failure
[1] calculate using the yield strength of longitudinal steel bar
[2] calculate using the ultimate strength of longitudinal steel bar

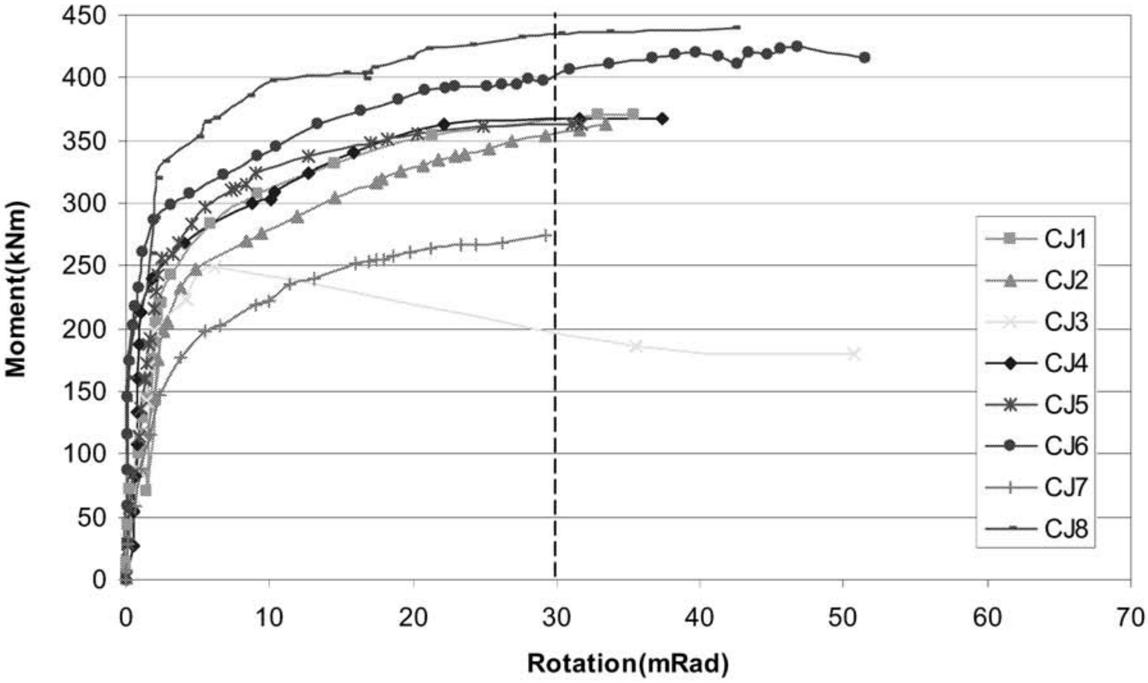

Figure 3 Moment vs. Rotation Curves

3. FINITE ELEMENT MODEL

The moment and the rotation capacity of the joints were studied using the 3-D finite element method. Using the general-purpose finite element package ABAQUS, a 3-D finite element model was built to simulate the behaviour of semi-rigid composite connection with precast hollowcore slabs. As shown in Figure 4, the model use three-dimensional solid elements to replicates the composite joint of the actual full scale test. The boundary condition and method of loading adopted in the finite element analysis followed closely to those used in the tests. The load was applied at the end of the beam as shown in Figure 4. Material nonlinearity was included in the finite element model by specifying the stress-strain curves of the material taken from the test specimens.

The comparison of the FE model with the test results are shown in Table 3 and 4 and typical moment rotation curve is shown in Figure 5 & 6. It can be seen that the model results has good agreement with the experiment data.

TABLE 3 COMPARISON OF MOMENT CAPACITY

Reference	**CJ1**	**CJ2**	**CJ3**	**CJ4**	**CJ5**	**CJ6**	**CJ7**	**CJ8**
Test result (kNm)	370	363	250	368	363	425	274	439
FE Model result (kNm)	407	402.9	253.7	383	398	437.6	292	475

TABLE 4 COMPARISON OF ROTATION CAPACITY

Reference	**CJ1**	**CJ2**	**CJ3**	**CJ4**	**CJ5**	**CJ6**	**CJ7**	**CJ8**
Test result (kNm)	35.4	33.5	6.1	37.4	31.7	46.8	30	42.3
Modelling result (kNm)	38.5	33.9	11.5	36	36.1	51.4	31.5	49.7

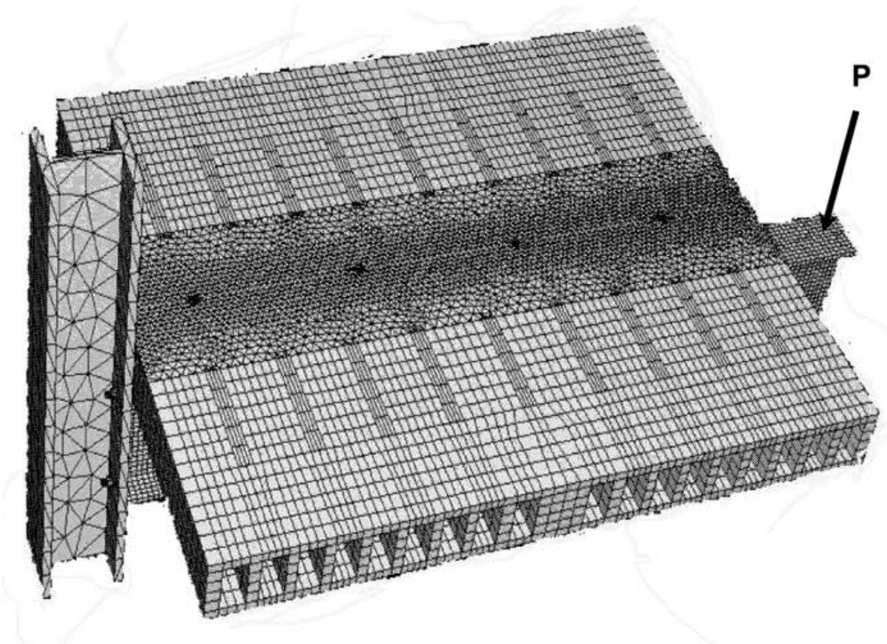

Figure 4 Finite Element Model of the Semi-Rigid Composite Joint

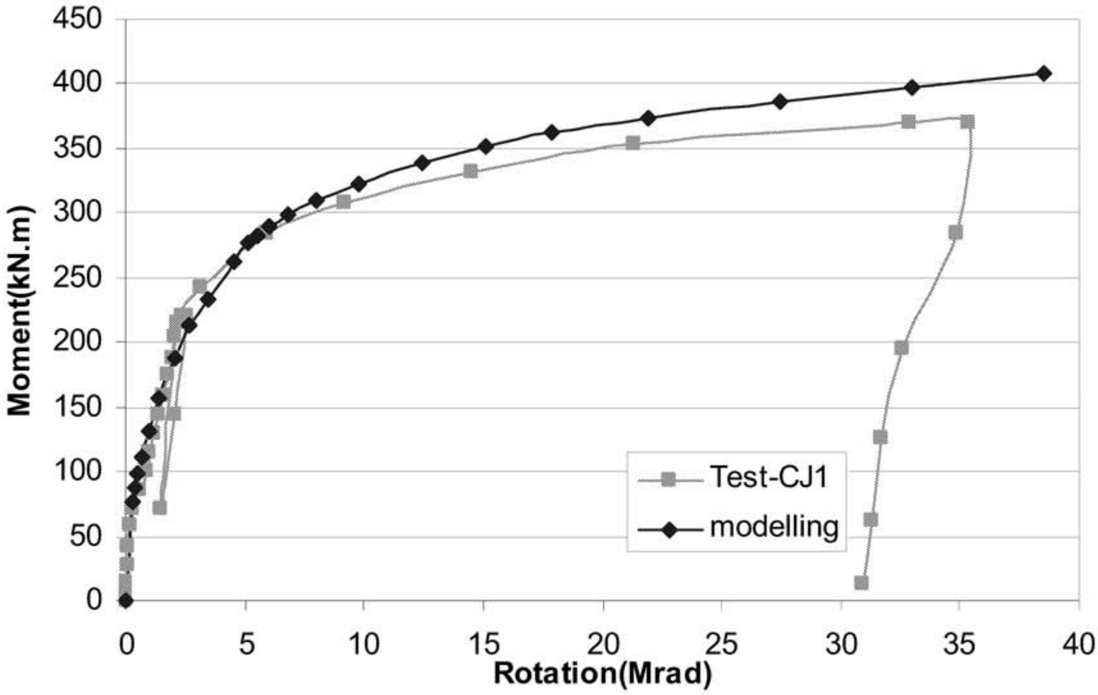

Figure 5 Comparison of Test CJ1 and the FE model

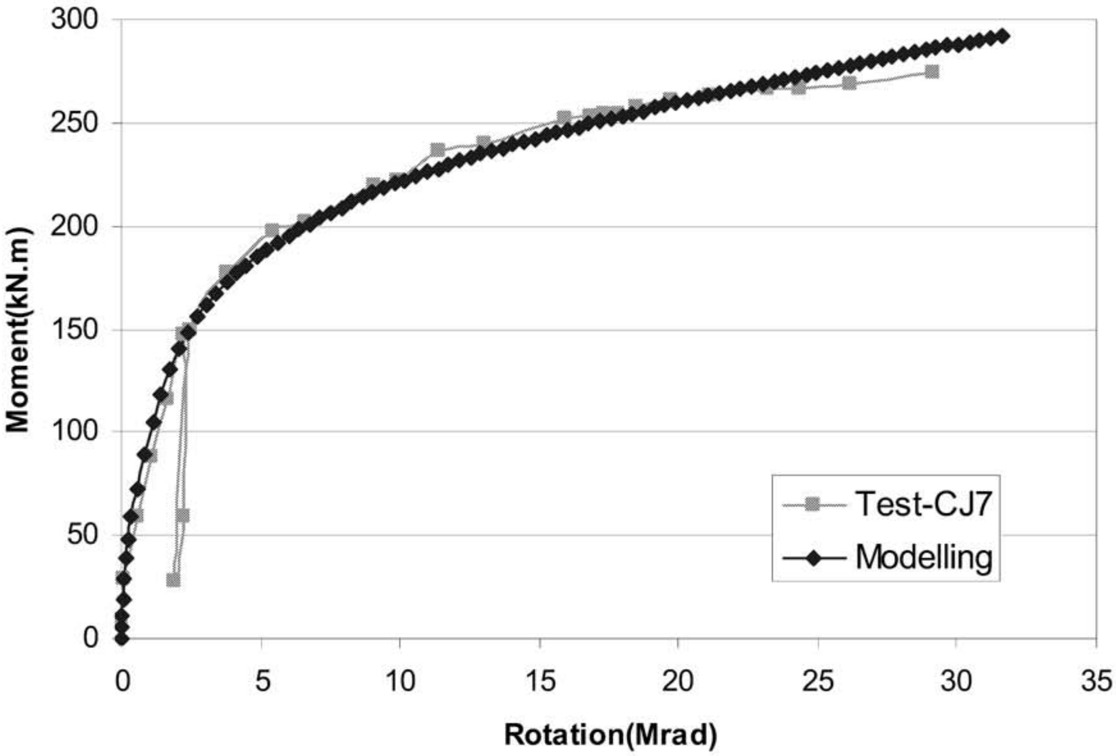

Figure 6 Comparison of Test CJ7 and the FE model

To better understand the structural behaviour of the joint, it is important to investigate the joints with systematic parametric studies. Table 5 shows the different parameters selected for FE analysis. Only one variable was changed at one group so as to asses its effect clearly. They are considered to be the most influential factors for the composite connections.

TABLE 5 VALUES OF PARAMETERS SELECTED FOR PARAMETRIC STUDIES

Variable	**Range of variable selected**
Spacing of shear stud	235mm, 535mm, 835mm, 1135mm
Endplate thickness	5mm, 10mm, 15mm, 20mm
Size of steel beam	457×191×89 UB, 406×140×39UB, 254×146×31UB
Column web	3.6mm, 7.2 mm, 14.4 mm
Precast slab thickness	150mm ,250mm, 400mm

4. ANALYTICAL MODEL

Base on the full scale tests and parametric studies, an analytical model to calculate the moment and rotation capacity for this type of connection is derived. Figure 7 describes the force transfer mechanism for the composite joint with flush end-plates composite connection.

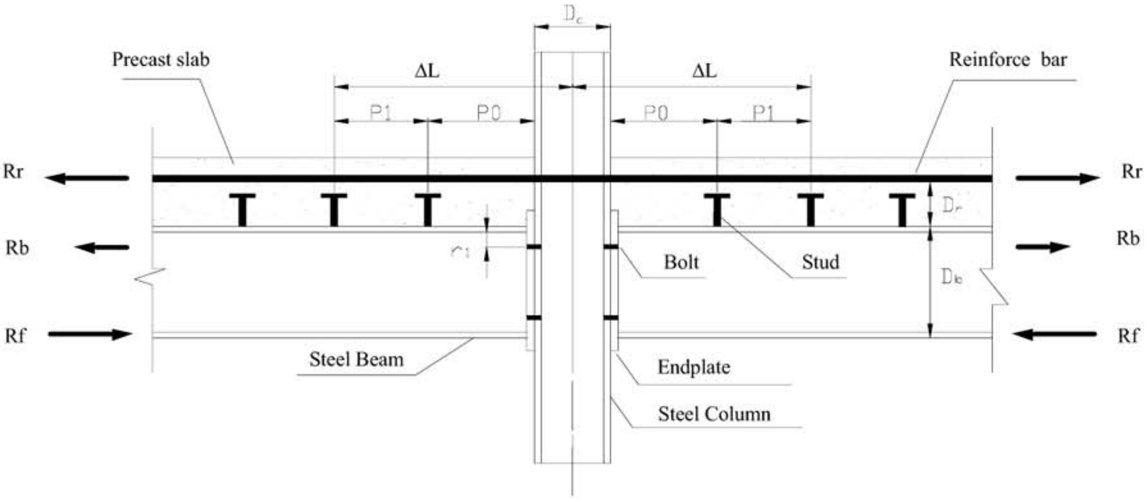

Figure 7 Load Transfer Mechanism for the Composite Joint

Tests result showed that the compression force transfer through direct bearing of the bottom flange of the beam. Due to strain hardening, it is possible for the bottom flange to resist compressive stresses of up to 1.2 times the yield strength. The tensile strength of the concrete is ignored as the tensile force of the slabs is relatively small, only the tensile strength of the longitudinal reinforcing bars was considered. A method to predict the moment capacity for this type of semi-rigid connection is proposed.

The proposed method assumes that:

For $R_f \geq R_b + R_r$,

where,
R_f= compressive resistance of the bottom flange of the steel beam,
R_r= tensile strength of the longitudinal reinforcement,
R_b= effective tensile resistance of the bolt group.

The moment resistance of the composite connection, $M_{c,Rd}$

$$M_{c,Rd} = R_r(D_b + D_r - 0.5t_f) + R_b(D_b - r_1 - 0.5t_f) \quad (1)$$

where,
D_b = the depth of the beam;
r_1 = the distance of the first row of bolts below the top of the beam;
D_r = the distance of the reinforcement above the top of the beam;
t_f = the flange thickness of the steel beam.

For $R_f < R_b + R_r$,

The neutral axis, $y_c = \dfrac{(R_r + R_b - R_f)}{t_w P_y}$

where,
t_w = the web thickness;
p_y = the design strength of steel section.

The moment resistance of the composite connection, $M_{c,Rd}$

$$M_{c,Rd} = R_r(D_b + D_r - 0.5t_f) + R_b(D_b - r_1 - 0.5t_f) - R_w \frac{y_c}{2} \quad (2)$$

where,
$R_w = y_c t_w p_y$

The comparison of the test results and the results from the proposed method above is shown in Table 6. The results showed that the moment capacity of the semi-rigid composite connections is dependent to the strength and the ability to mobilize the longitudinal reinforcing bars. The influential factor for their mobilization is depending on the degree of the shear connection, which is determined by the number and the capacity of the shear studs in the hogging moment region.

TABLE 6 COMPARISON OF MOMENT CAPACITY

Reference	CJ1	CJ2	CJ3	CJ4	CJ5	CJ6	CJ7	CJ8
Test result (kNm)	370	363	250	368	363	425	274	439
Analytical model (kNm)	**365.8**	**365.8**	**284.5**	**365.0**	**366.6**	**422.3**	**274.0**	**446.7**

The available rotation capacity is dependent on the mode of failure for this form of construction. For the composite joints, the deformation is provided by yielding and inelastic elongation of the slab reinforcement and slip of the shear connectors. An analytical method is proposed for predicting the rotation capacity for this form of composite joints:

$$\phi_j = \frac{\Delta_r}{D_b + D_r} + \frac{S}{D_b} \quad (3)$$

In order to determine the elongation of the longitudinal steel bar, the effective deformation length of the longitudinal rebar, ΔL need to be determined first. From the tests result, it showed that the yielding of the longitudinal reinforcement only occurred at the distance between the centre line of the column and the second stud if the distance between the first stud and the column flange is less than 900 mm. The strain in the other part of the rebar is relatively small. Hence, the effective deformation length is assumed to be P_0+P_1+ $D_c/2$ as shown in Figure 7 until the ultimate stress is reached. This demonstrates that position of the headed studs played an important role in the rotation capacity of the composite connections.

The deformation capacity is influenced not only by the effective deformation length and ductility of the reinforcing bars in the region near the joint but also by tension stiffening. When the concrete is crack and yielding of the reinforcement occurred, the effect of tension stiffening increases significantly. This is because the bond between concrete and reinforcement lowers the strain away from the cracks as shown in Figure 8.

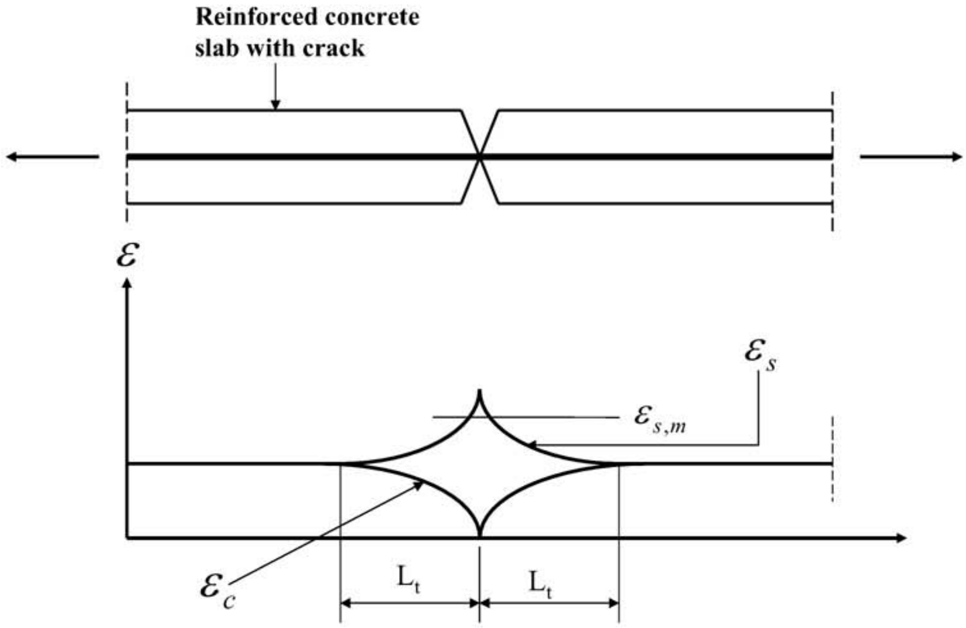

Figure 8 Strain in Cracked Reinforced Concrete

The stress-strain relationship for embedded reinforcement provides a higher stiffness and rupture at a lower ductility than the reinforcement alone. The ultimate mean strain, ε_{smu} in embedded reinforcement, with the tension stiffening effect taken into account, which will arise from the crack over the transmission length, L_t which the bond has broken down.

The ultimate mean strain, ε_{smu}

$$\varepsilon_{smu} = \frac{1}{2}(\varepsilon_{su} + \varepsilon_{sy}) \tag{4}$$

where,

ε_{su} is the ultimate strain of the reinforcement

ε_{sy} is the yield strain of the reinforcement

The transmission length, L_t

$$L_t = \frac{k_c f_{ctm} \phi}{4 \tau_{sm} \rho} \tag{5}$$

where,

f_{ctm} is the tensile strength of concrete

ρ is the longitudinal reinforcement ratio where $\rho = \frac{A_s}{A_c}$

A_s is the area of the longitudinal bar

A_c is the area of the effective concrete slab, for composite precast hollowcore slabs, the region of the in-situ concrete infill is used.

k_c is a coefficient that allows for the self-equilibrating stresses and the stress distribution in the slab prior to cracking where $k_c = \dfrac{1}{1+\dfrac{h_{cs}}{2z_0}}$

h_{cs} is the thickness of the precast slab

z_o is the vertical distance from the centroid of the uncracked unreinforced concrete flange to the neutral axis of uncracked unreinforced composite section, which is calculated ignoring the reinforcement and using the modular ratio for short-term effects, E_s/E_{cm}.

ϕ is the diameter of the rebars

τ_{sm} is the average bond stress along the transmission length and is taken as $1.8\, f_{ctm}$

For full shear connection, the formula for calculating the elongation of the longitudinal rebar, Δ_r is defined as follows:

For $\rho \leq 1.0$ %,

$$\Delta_r = \left(\frac{D_c}{2} + 2L_t\right) \times \varepsilon_{smu} \tag{6}$$

For $\rho > 1.0$ % and $P_0 \leq L_t$

$$\Delta_r = \left(\frac{D_c}{2} + 2L_t\right) \times \varepsilon_{smu} + (P_1 - L_t) \times \varepsilon_y \tag{7}$$

For □> 1.0 % and $P_0 > L_t$

$$\Delta_r = \left(\frac{D_c}{2} + P_0 + L_t\right) \times \varepsilon_{smu} + (P_1 - L_t) \times \varepsilon_y \tag{8}$$

For partial shear connection, the ultimate mean strain, ε_{smu} is taken at the on set of strain hardening if yielding of the longitudinal reinforcement can be achieved. Otherwise, ε_{smu} is taken as the yield strain, ε_y. The stress strain curve of the longitudinal rebar is shown in Figure 9.

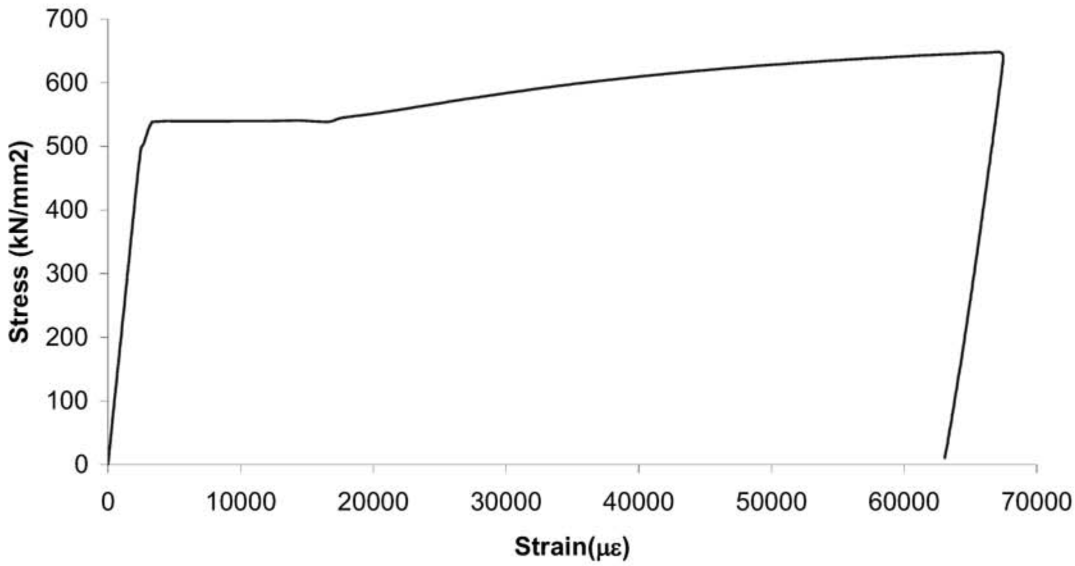

Figure 9 Stress vs. Strain Curve of the Reinforcing Bar

The slip of the shear connectors can be taken directly from the standard push test. Figure 10 shows the load vs. slip curve of the 19mm headed shear stud. The correspondence shear force of the stud is taken as

$$F_s = \frac{A_s f_y}{n} \tag{9}$$

where,

$A_s f_y$ is the maximum yield strength of the longitudinal reinforcement;
n is the total numbers of shear connector.

The comparison of the test results and the results from the analytical method for rotation capacity above is shown in Table 7. Results showed a reasonable agreement between the test results and the analytical method with the exception of CJ3 which is due to premature failure of the slabs.

TABLE 7 COMPARISON OF ROTATION CAPACITY

Reference	**CJ1**	**CJ2**	**CJ3**	**CJ4**	**CJ5**	**CJ6**	**CJ7**	**CJ8**
Test result (mRad)	35.4	33.5	6.1	37.4	31.7	46.8	30	42.3
Analytical method (mRad)	29.1	31.3	18.7	30.0	28.7	43.4	27.9	53.6

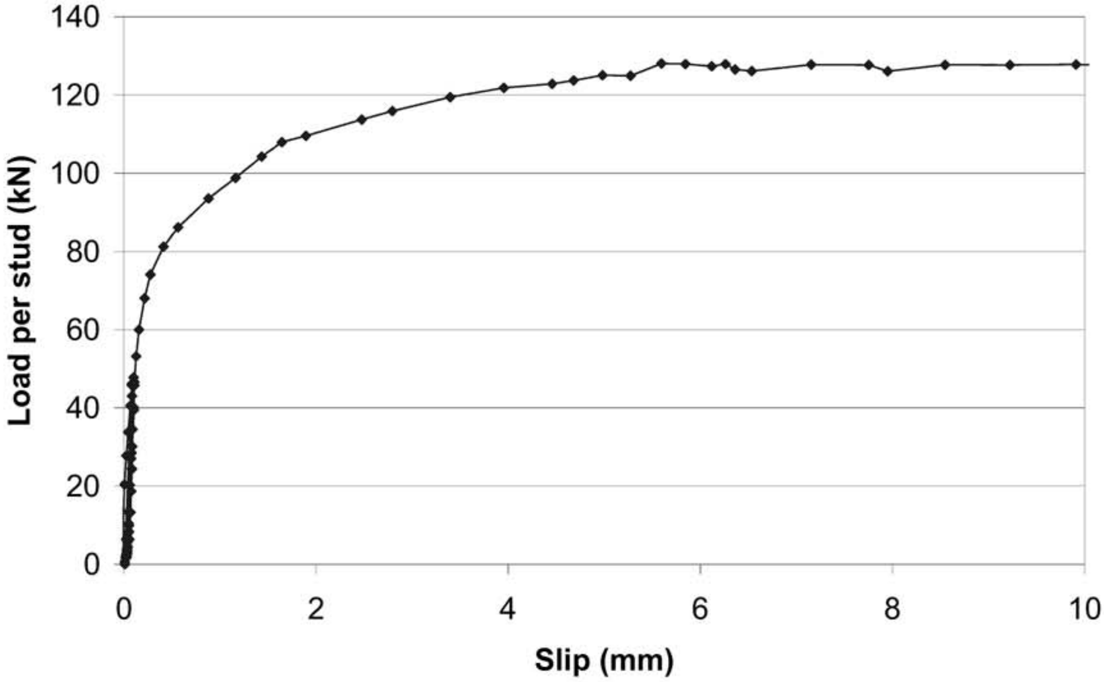

Figure 10 Load vs. Slip of 19mm Headed Shear Stud

5. CONCLUSIONS

Tests program designed to study the moment and rotation capacity of the composite joints with precast hollowcore slabs has been described as well as a FE model was presented to investigate the structural behaviour of the composite joints. The comparison with the test results showed that the proposed model can accurately represent the overall behaviour of the composite joints. Based on the intensive parametric study and experimental results, an analytical method to calculate the moment and rotation capacity of the composite joints with precast hollowcore slabs were derived and good agreement has been obtained when compare with the tests result. The results show that the proposed analytical method is adequate to use for designing this form of composite joints.

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ANCHOR DESIGN FOR DYNAMIC LOADS – EARTHQUAKES AND SHOCK

Dr J. Kunz

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Helmut Gassner, Hilti Corporate Research, Liechtenstein

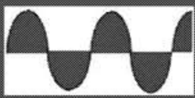
Basics

Dynamic Loads

Safety Concept

Behaviour of Prestressed Fastenings

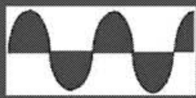
Design for Fatigue Loads



Fatigue Testing

Influence of Permanent Loads

Practical Design Method



HILTI Hilti BA Fastening & Protection
Anchor Design for Dynamic Loads - part 1: basic principles and fatigue

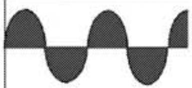
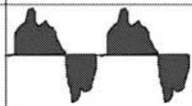
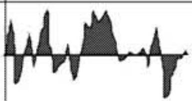
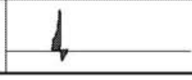
Basics: Possible Dynamic Actions

characteristic of dynamic actions:

induced accelerations activate forces of inertia and damping

Classification	Fatigue	Seismic	Shock
Frequency of appearance	$10^4 < n < 10^8$	$10^1 < n < 10^4$	$1 < n < 20$
Rate of strain	$10^{-6} < \varepsilon' < 10^{-3}$	$10^{-5} < \varepsilon' < 10^{-2}$	$10^{-3} < \varepsilon' < 10^{-1}$
Examples	•Traffic loads •Machines, cranes •Ventilators •Wind, Waves	•Earthquakes •Artifical earthquakes	•Explosions •Abrupt Struture failure •Crash Barriers

Basics: Possible Dynamic Actions

Action	Chronological Sequence		Possible Cause
harmonical (alternating load)		sinusoidal shape	rotating machines
harmonical (swelling load)		sinusoidal shape	
periodic		periodic with any shape	textile machines, punching machines
stochastic		non periodic with any shape	earthquake, rail and road traffic
impact type		short time of action	impact, explosion, rapidly closing valves

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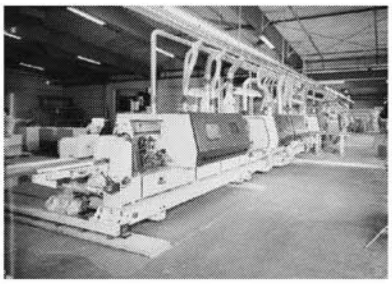
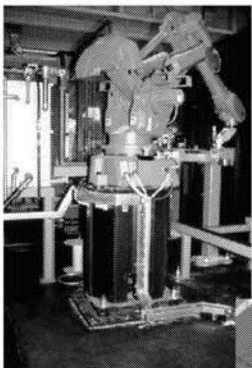
Anchor Design for Dynamic Loads - part 1: basic principles and fatigue

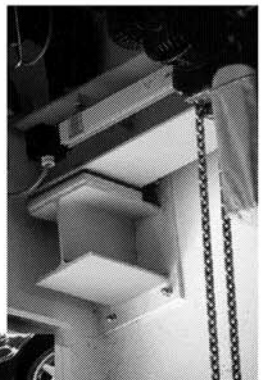
Fatigue: Examples

Machine fixing
(e.g. pumps, ventilators,
punching machines, etc.)





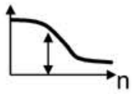
Robot fixing
(e.g. in automobile
industry, etc.)



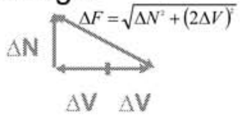
Fixing of cranes,
hydro-jacks, etc.)

Fatigue: Main Challanges

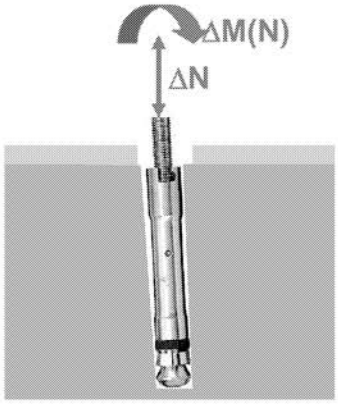
1. Materials: Loss of steel and concrete strength



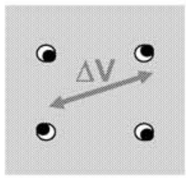
2. Determination of loads during life time

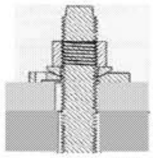


3. No anchor is 100% vertical
➔ most crucial bending in anchor



4. Clearance hole is bigger than anchor ∅
➔ movement of baseplate if friction has been overcome

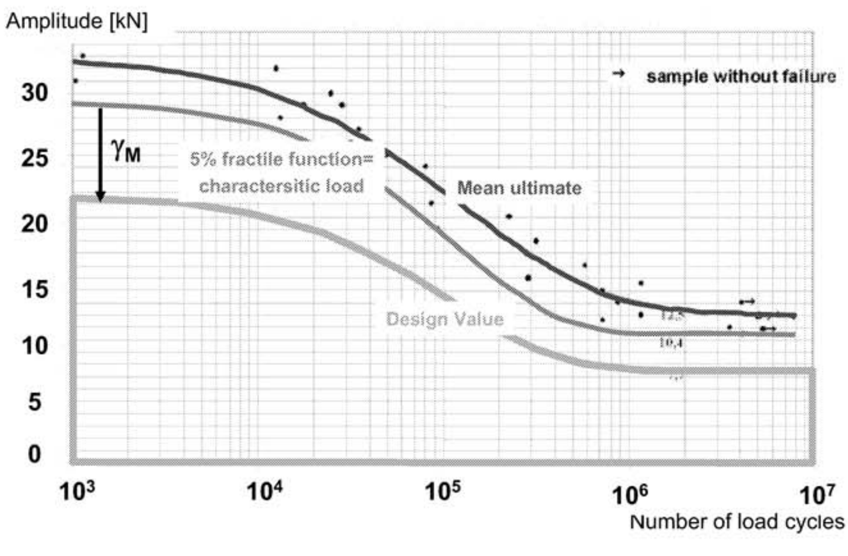




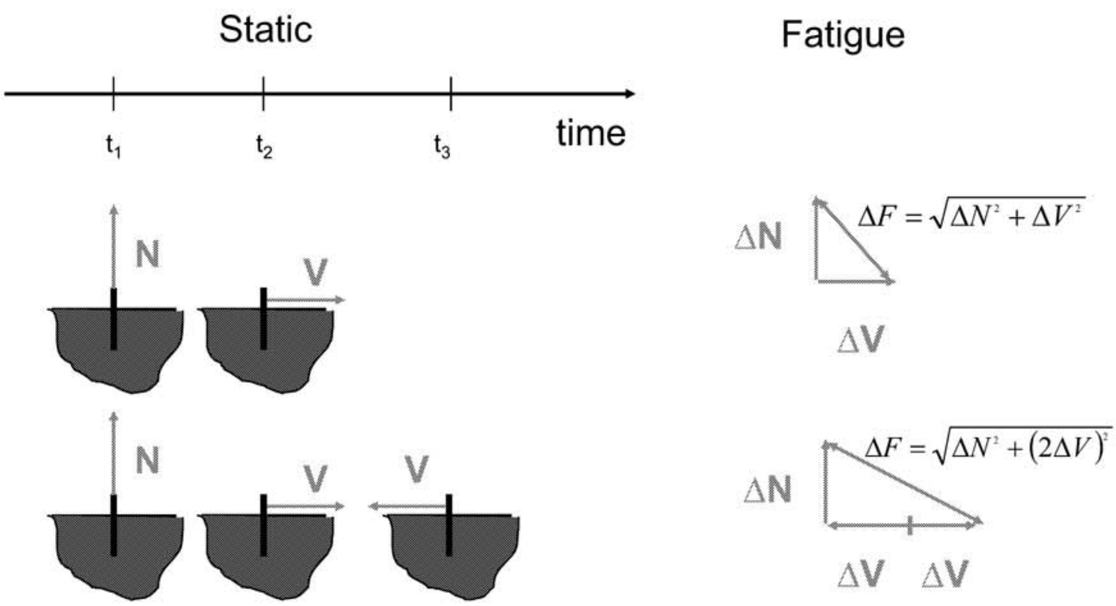
5. Loosening of nuts

Fatigue: 1. Materials

- loss of steel and concrete strength with increasing number of load cycles
- often used: Wöhler Curves



Fatigue: 2. Determination of Fatigue Relevant Loads



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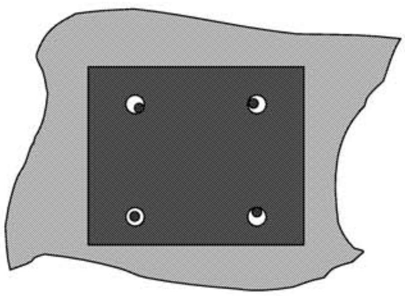
Anchor Design for Dynamic Loads - part 1: basic principles and fatigue

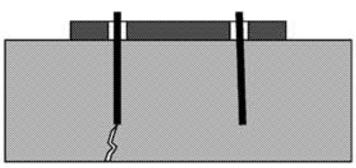
Fatigue: 3. Bending / 4. Clearance / 5. Loose nuts



Different anchors in anchor group have different stiffness, due to

- verticality
- cracks

Load redistribution
stiffer anchors are higher loaded and therefor will fail earlier



Group factors for multiple fastenings different for tension and shear, anchor type related

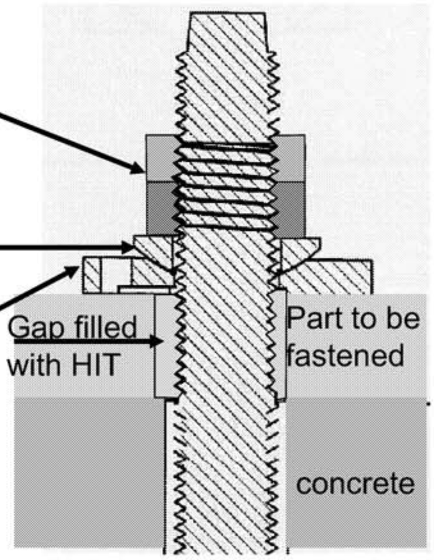
Fatigue: 3. Bending / 4. Clearance / 5. Loose nuts

Function of Dynamic Set

Nut and additional locknut
avoids loosening of whole system and ensures pretensioning and clamping

Spherical washer
avoids bending moments in anchor, when not 100% vertical set

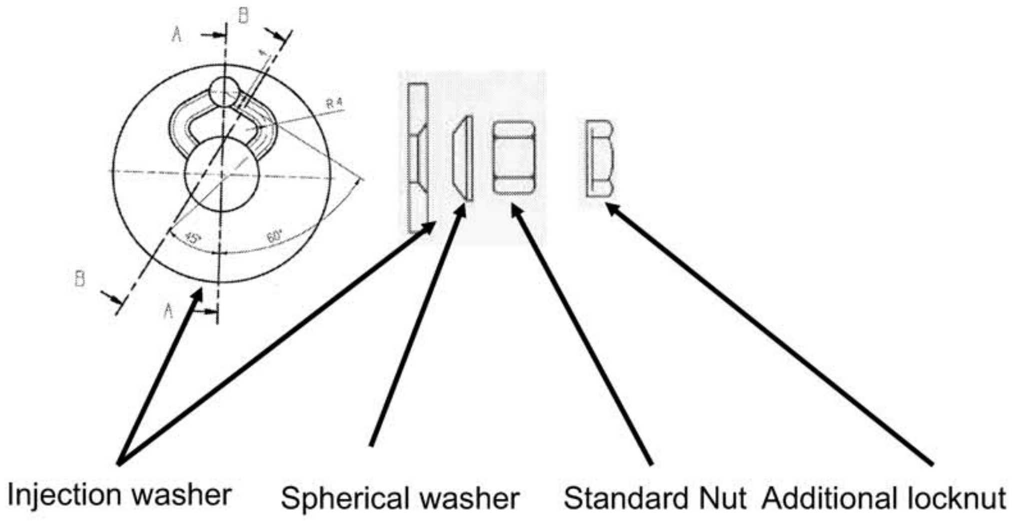
Injection washer
necessary to fill up clearance hole with HIT ➔ no gap between part to be fastened and anchor



Fatigue: 3. Bending / 4. Clearance / 5. Loose nuts

Dynamic Set

For a perfect fatigue fastening the “Dynamic Set” has to be used



Fatigue: Design
Safety Concept According to Eurocode

dynamic action	fatigue	shock	earthquake
characteristic resistance under dynamic action	$R_{k,fatigue}$	$R_{k,shock}$	$R_{k,earthquake}$
partial safety factor (resistance)	$\gamma_{M,fatigue}$	$\gamma_{M,shock}$	$\gamma_{M,earthquake}$
design value of resistance under dynamic action		$R_d = R_k / \gamma_M$	
design		$R_d \geq S_d$	
design value of dynamic action		$S_d = S_k * \gamma_F$	
partial safety factor (action)	$\gamma_{F,fatigue}$	$\gamma_{F,shock}$	$\gamma_{F,earthquake}$
characteristic value of dynamic action	$S_{k,fatigue}$	$S_{k,shock}$	$S_{k,earthquake}$

Hilti BA Fastening & Protection
Anchor Design for Dynamic Loads - part 1: basic principles and fatigue

Fatigue: Design for Anchors - Hilti method

Static check

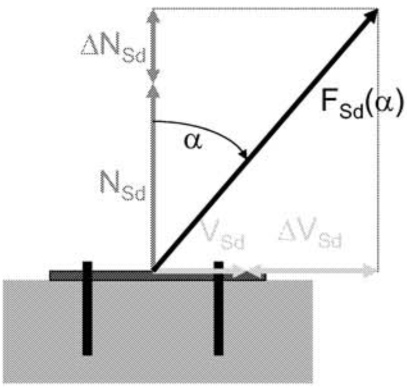
Acc. to standard design methods e.g. cc-method

Tension

$$\min(N_{Rd,c}, N_{Rd,s}) \geq N_d + \Delta N_d$$

Shear

$$\min(V_{Rd,c}, V_{Rd,s}) \geq V_d + \Delta V_d$$



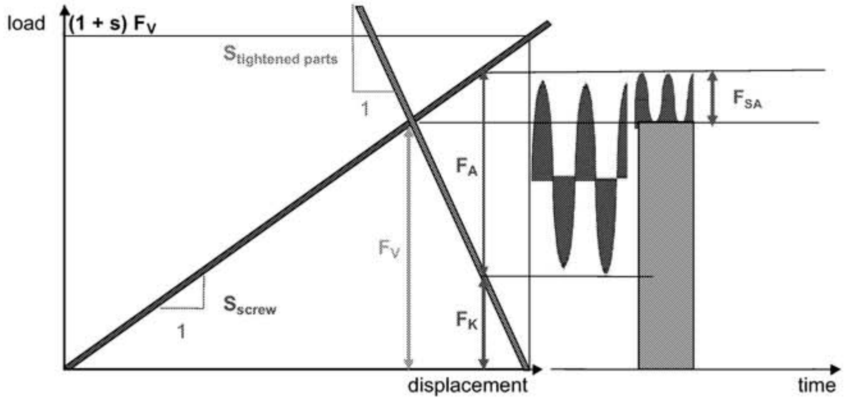
Combined load

$$F_{Sd}(\alpha) = F_{Rd} = \left[\left(\frac{\cos\alpha}{N_{Rd}}\right)^{1.5} + \left(\frac{\sin\alpha}{V_{Rd}}\right)^{1.5}\right]^{-\frac{2}{3}}$$

Fatigue: Design for Anchors - Hilti method

Pretension influence

The prestressing F_V reduces the extern dynamic working load F_A to the part, which relevant to fatigue in the screw F_{SA}



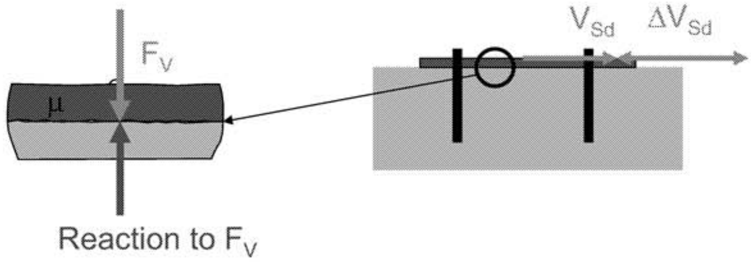
F_A	extern working load	F_V	prestressing force
F_K	clamping force	S_{screw}	stiffness of screw
F_{SA}	Force in screw relevant to fatigue	$S_{tightened\ parts}$	stiffness of tightened parts

Hilti BA Fastening & Protection
Anchor Design for Dynamic Loads - part 1: basic principles and fatigue

Fatigue: Design for Anchors - Hilti method

Friction is characterised by:
friction factor μ
prestressing force F_V

$$F_R = F_V * \mu$$

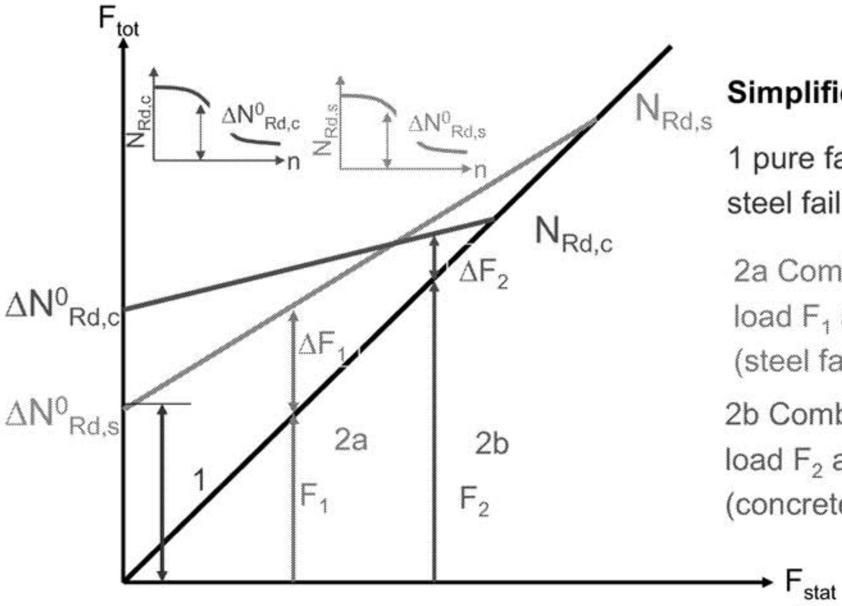


Simplified assumption:

- if $F_R > V_{Sd} + \Delta V_{Sd}$ then shear on anchor =0
- if $F_R < V_{Sd} + \Delta V_{Sd}$ then shear on anchor = $V_{Sd} + \Delta V_{Sd}$

Fatigue: Design for Anchors - Hilti method

Influence of # load cycles, static and fatigue relevant load



Simplified Goodman Chart

1 pure fatigue load: in general steel failure for n>10'000

2a Combination of static load F_1 and fatigue load ΔF_1 (steel failure)

2b Combination of static load F_2 and fatigue load ΔF_2 (concrete failure)

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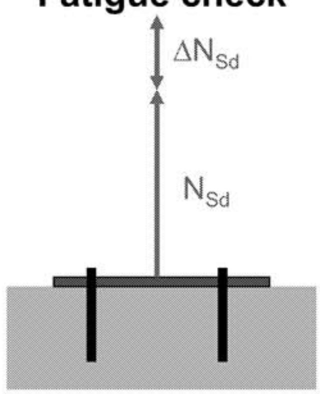
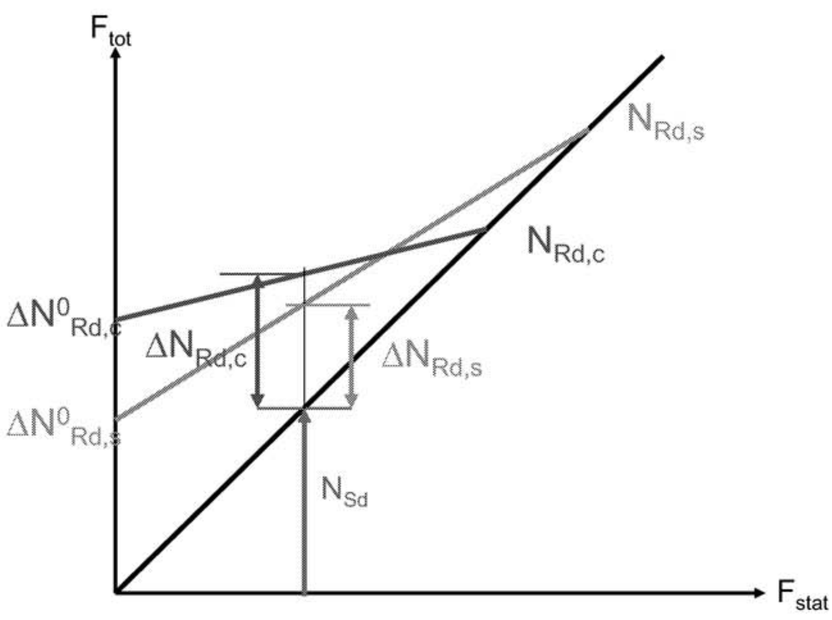
Anchor Design for Dynamic Loads - part 1: basic principles and fatigue

Fatigue: Design for Anchors - Hilti method

Fatigue check



$$\frac{\Delta N_{Sd}}{\Delta N_{Rd}} \leq 1.0$$



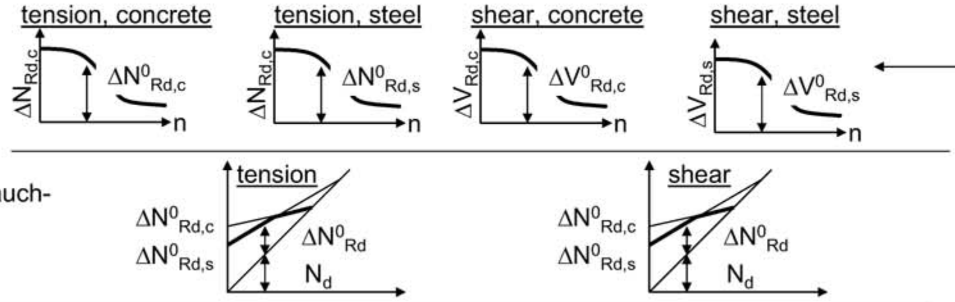
Fatigue: Design for Anchors - Hilti method

Design Procedure Hilti: detailed calculation (HIDU/HAP-program)

resistance

a) Wöhler-charts

b) lin. Weyrauch-charts



fatigue check	$\Delta N_{Rd} \geq \Delta N_d$	$\Delta V_{Rd} \geq \Delta V_d$
static check	$\min(N_{Rd,c}, N_{Rd,s}) \geq N_d + \Delta N_d$	$\min(V_{Rd,c}, V_{Rd,s}) \geq V_d + \Delta V_d$
friction check		$\mu \cdot N_{kd,\min} \geq V_d^a + \Delta V_d^a$

pretension: None installation torque. → min. tension in screw: stat. load $\mathbf{N_d}$ | tens. amplitude fatigue relevant $\mathbf{\Delta N_d}$ | min. clamp. force $\mathbf{N_{kd,min}}$

Acting loads static: $\mathbf{N^a_d}$; $\mathbf{V^a_d}$, alternating: $\mathbf{\Delta N^a_d}$, $\mathbf{\Delta V^a_d}$ friction: μ nbr. cycles: **n**

Hilti BA Fastening & Protection
Anchor Design for Dynamic Loads - part 1: basic principles and fatigue

Fatigue: Design for Anchors - DIBt1) - method
Simplified Assumptions

- determination of fatigue loads (all loads are fatigue relevant)
- for all number of load cycles n≥ 10'000 ➔ n=2'000'000
- load safety factor γ_F=1.0
- group factor $\gamma_{F,N}$=1.3 for HDA Tension and $\gamma_{F,V}$= 1.2 for HDA Shear
- no stand-off fixings
- calculation according to same rules as in cc-method
- same factors for reduced spacing and edge distance
- ***reduced design values for steel and concrete***
- ***other interaction of tensile and shear force check***

1) Deutsches Institut für Bautechnik: German Institute for Construction Technology

Fatigue: Design for Anchors - DIBt1) - method

Main Differences Static - Fatigue design

Static Design (cc-method)	Dynamic Design
Load safety factors γ_G =1.35 acc. Eurocode γ_Q =1.5 acc. Eurocode	Load safety factors γ_F=1.0 Group factors $\gamma_{F,N}$, $\gamma_{F,V}$ =1.0 for single anchors; >1.0 for group fastenings
Characteristic resistances related to static concrete and steel strength $N_{Rk,s}$, $N_{Rk,c}$, $V_{Rk,s}$, $V_{Rk,c}$	Reduced characteristic resistances related to fatigue concrete and steel strength $\Delta N_{Rk,s}$, $\Delta N_{Rk,c}$, $\Delta V_{Rk,s}$, $\Delta V_{Rk,c}$ depending on number of load cycles

Different material safety factors $\gamma_{M,s}$ $\gamma_{M,c}$

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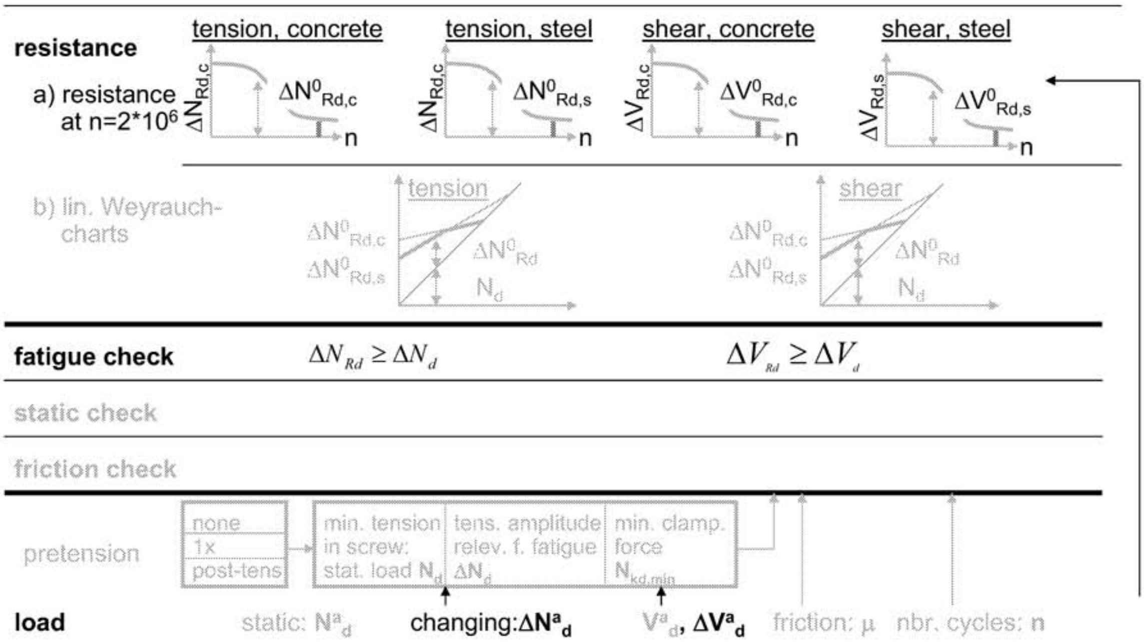
Anchor Design for Dynamic Loads - part 1: basic principles and fatigue

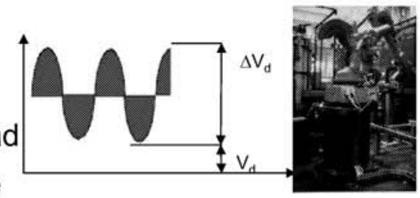
Fatigue: Design for Anchors - DIBt1) - method

Design Procedure DIBt: Simplified Calculation (DIBt approval)



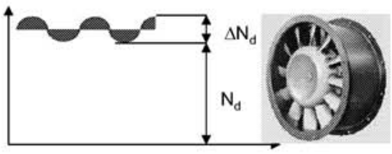
Fatigue: Summary

Resistance depends on



• ratio between fatigue relevant load and static load

small static load ➔ big fatigue resistance
(e.g. machines, etc.)



big static load ➔ small fatigue resistance
(e.g. ventilators)

• number of load cycles

• single anchor or group fastening (load transfer from anchors with small stiffness, e.g. in a crack, to those with higher stiffness)

• prestressing improves resistance against fatigue

relaxation of steel and creeping of concrete reduces prestressing

at cracks in concrete the prestressing force vanishes

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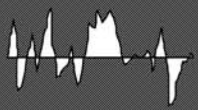
Earthquake

Impact on Fasteners
Cracks, Ductility
Testing, Research



Shock

Impact on Fasteners
Testing
Resistance of Anchors

Special Application: Nuclear Power Plants

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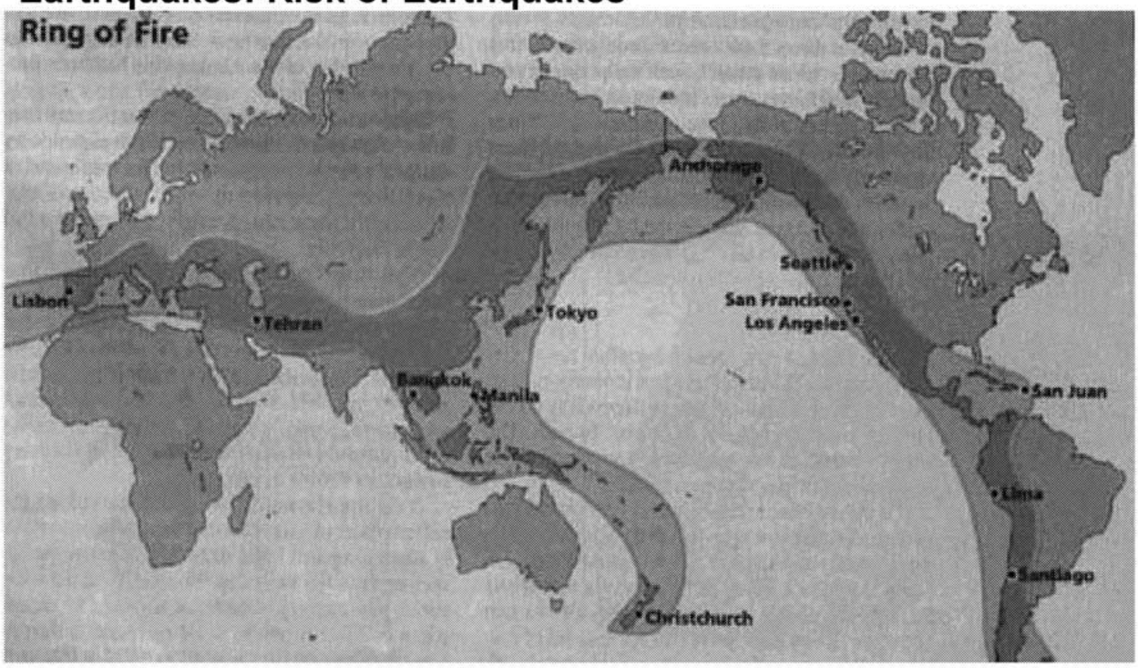
Anchor Design for Dynamic Loads - part 2: earthquakes and shock

Earthquakes: Risk of Earthquakes



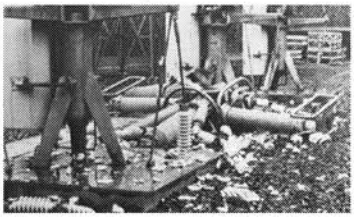
Earthquakes: Examples

Typical Fastening Problems (structural and non structural)

Suspended Ceilings
San Francisco Airport
Loma Prieta earthquake
1989

Toppled and damaged
furniture,City Hall Kobe, 1995

Falling wall panels
Public Building, Kobe, 1995



Damages in a Switchyard
Managua, 1972

Destroyed column fixing
San Francisco, 1989

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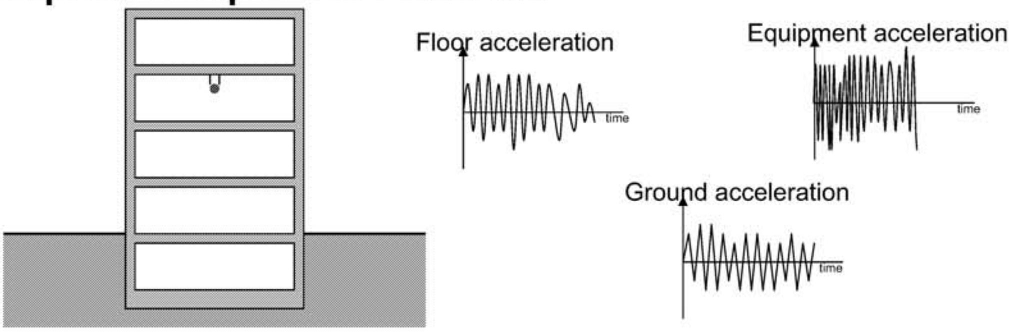
Anchor Design for Dynamic Loads - part 2: earthquakes and shock

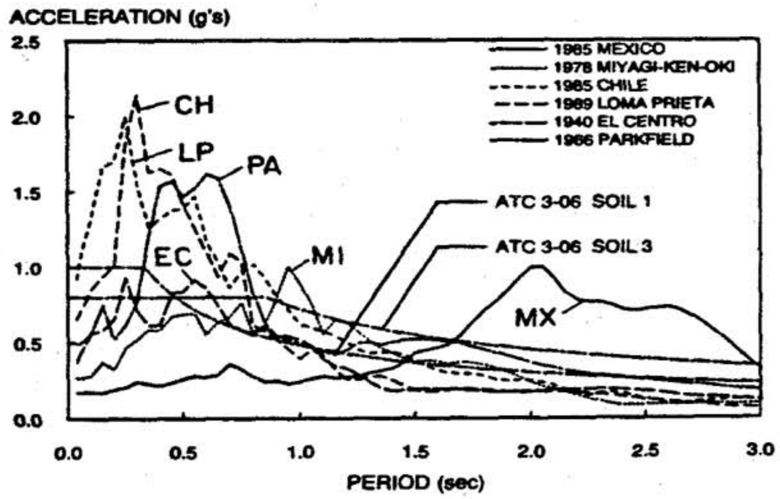
Earthquakes: Impact on Fasteners



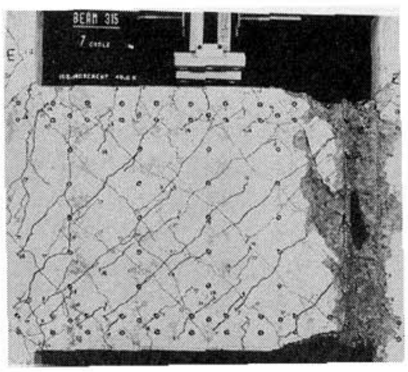
- Earthquakes cause acceleration of the ground
- Acceleration is transferred to the building
- The Building transfers the acceleration to the equipment
- The huge amount of influencing factors makes it impossible to predict the seismic loads accurately
- Loading on the equipment and from the equipment to the fastener can only be estimated

Earthquakes: Impact on Fasteners

Accelerations Measured During Different Earthquakes



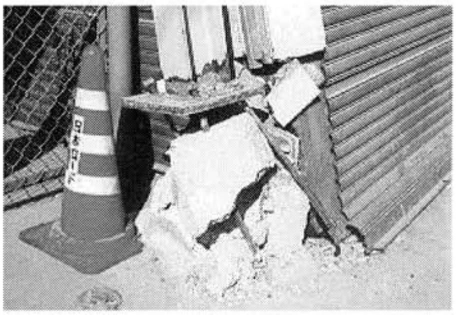
Earthquakes: Cracks Caused by Earthquakes

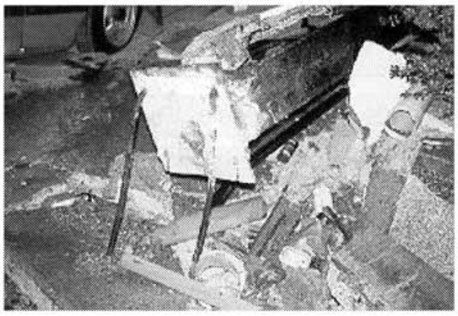


- Crack widths are not predictable.
- Load bearing capacity of anchors in the cracked concrete is not predictable with precision -> high safety factor
- Importance to use anchors suitable for cracked concrete

Earthquakes: Ductility

- Stiff structures lead to high earthquake loads.
- Ductility absorbs energy -> deformations, no failure
- Anchors are not the right place to bring in ductility
- Anchor design has to be done respecting the whole structure





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Anchor Design for Dynamic Loads - part 2: earthquakes and shock

Earthquakes: Design

Example: Uniform Building Code (USA)

•estimation of horizontal load (static)

$$\boxed{F_p = Z \cdot I \cdot C_p \cdot W_p}$$

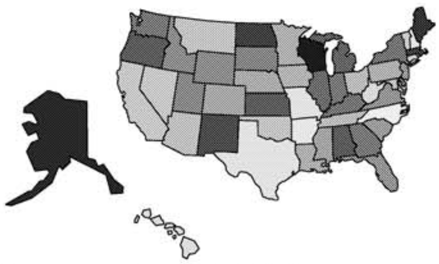
F_P	static horizontal load
Z	factor for seismic zone (from code)
I	factor for acceleration
C_P	factor for stiffness of structure
W_P	mass of element / equipment

•resistance: safety factor 4

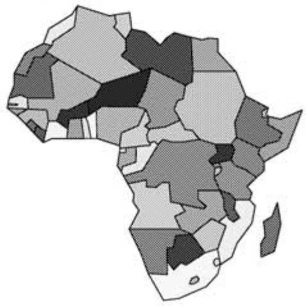
“fasteners shall be designed for **four times** the forces determined“

Earthquakes: Testing of Anchors

Many different test procedures according to national regulations



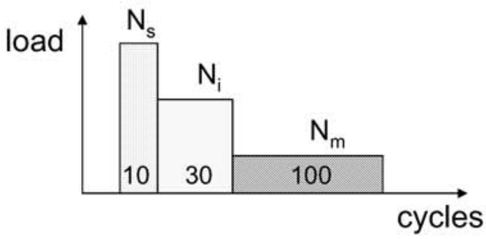
Excerpt from ICBO ES Acceptance Criteria (seismic method 6.2.7.2):



- Tension: pulsating sinusoidal seismic cycle
- shear: alternating sinusoidal seismic cycle
- frequency 0.1 to 0.2 Hertz
- at least 5 anchors
- shallowest and deepest embedment depth

Earthquakes: Testing of Anchors - Tension

- Maximum load N_s: 1.5∗tension value for which recognition is desired
- The minimum load value shall not be smaller than 5% of F_u
- Uncracked concrete



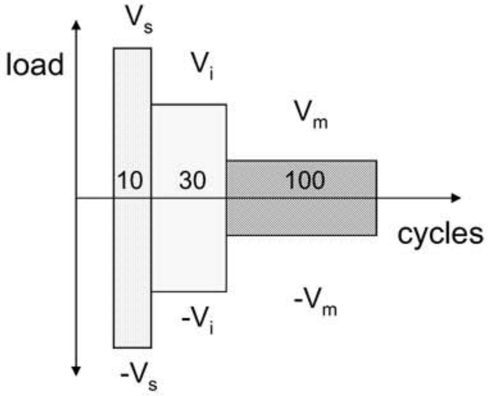
N_s: maximum tension test load

N_i: a load midway between N_s and N_m

N_m: 1/4 of the average ultimate tension load T_{ref}

Earthquakes: Testing of Anchors - Shear

- Maximum load V_s: 1.5∗shear value for which recognition is desired
- The value for which recognition is desired shall not be larger than 133.33 percent of the allowable static loads under the same conditions
- Uncracked concrete



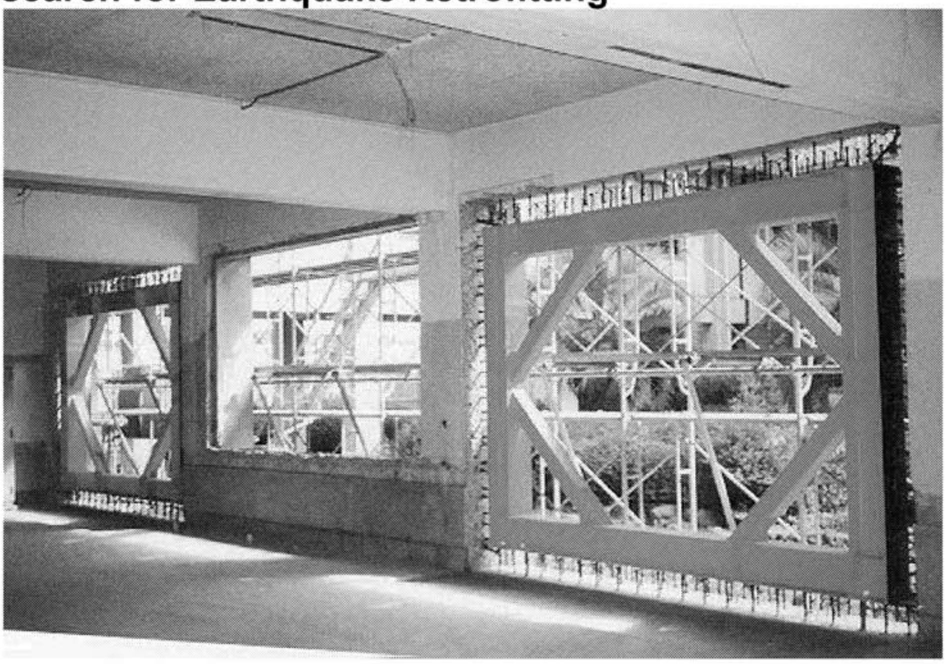
V_s: maximum shear test load

V_i: a load midway between V_s and V_m

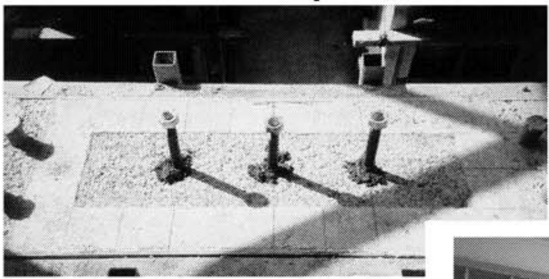
V_m: 1/4 of the average ultimate tension load T_{ref}

Anchor Design for Dynamic Loads - part 2: earthquakes and shock

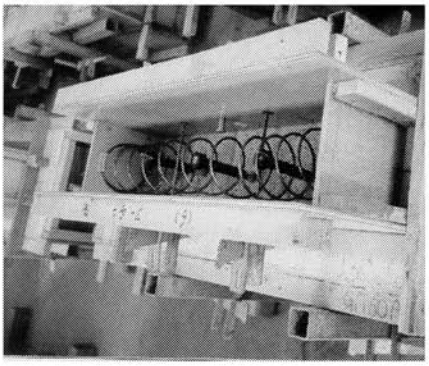
Research for Earthquake Retrofitting

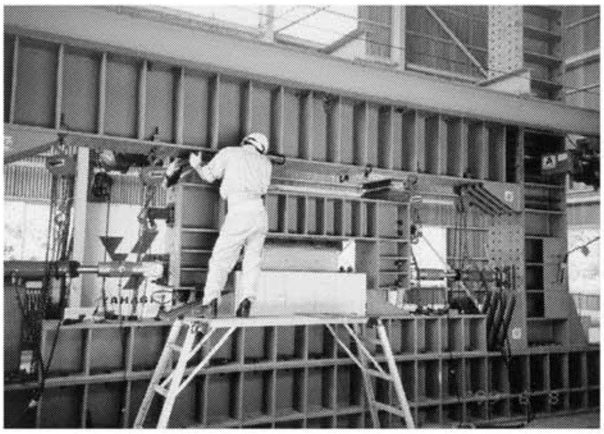


Research for Earthquake Retrofitting



experimental setup at
Shibaura Institute of Technology
Tokyo





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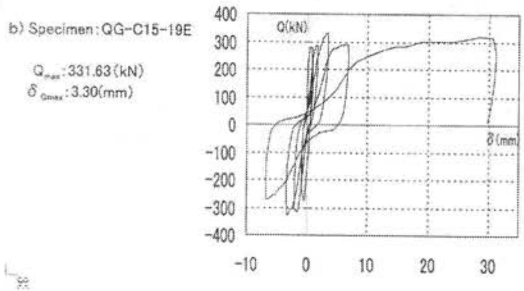
Anchor Design for Dynamic Loads - part 2: earthquakes and shock

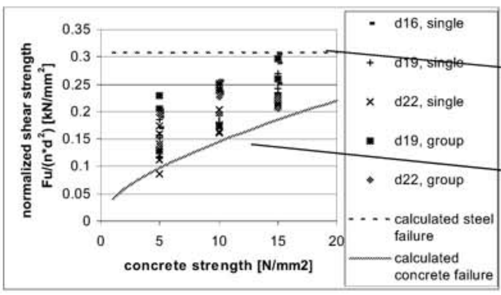
Research for Earthquake Retrofitting



„The comparison of these results with the customarily used design equations...given by JBDPA indicates that:

when anchor rods are imbedded into the ordinary low-strength concrete, there will be no shortage of resistance.“

Prof. Y. Yamamoto, Research Report 1999



Shear Resistance According to EC4

steel shear failure: $P_{Rk} = 0.8 f_u \dfrac{\pi \cdot d^2}{4}$

concrete bearing pressure: $P_{Rk} = 0.29 d^2 \sqrt{f_{ck} \cdot E_{cm}}$

Shock: Impact on Fasteners

- vehicle crash (on crash barriers or other structures)
- emergency stop of lift
- emergency installations
- explosions
- fast shutting of pipe valves

Shock loads are not “normal” so in some cases (e.g. explosion impact) a damage of the structure (or the anchor) is accepted but not a collapse

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Anchor Design for Dynamic Loads - part 2: earthquakes and shock

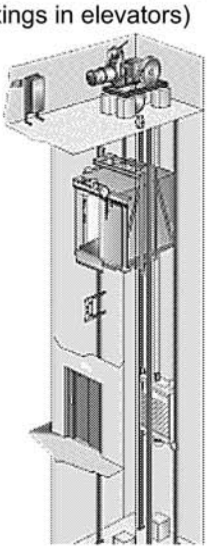
Shock: Examples

Typical Fastening Problems (structural)

Crash barrier fixing on concrete
(e.g. on bridges, etc.)

Explosions
(e.g. installations in civil shelters)

Emergency Installations
(e.g. emergency stop fixings in elevators)



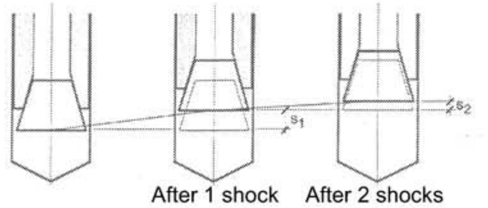
Shock: Testing

- single anchor in cracked concrete (**w=1.0mm!**) - concrete quality ~C20/25
- Load: 1.5*N_{rec} for “older” anchors / N_d for anchors tested according to ETA
- ETA or a comparable approval (e.g. DIBt) is imperative base
- two shocks for each anchor

Criteria:

1. No pullout after 2 shocks, s_{tot} <10mm

2. Stable slip development
   $s_2 < s_1$

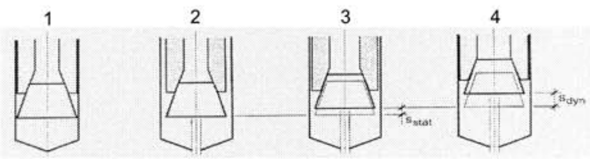


Shock: Resistance of Anchors

Anchors are set in the closed cracks. Cracks are opened to w=1.0mm after the installation of the tightening torque.

Slipping of the anchors under the shock load is normal because of the subsequent opening of the crack.

Slipping of expansion anchors is in general bigger than in undercut systems. Bonded anchors are very sensitive for subsequent opened cracks - up to now only HVZ succeeded in the BZS-Shock approval.



1. Installation
2. Opening of crack
3. Slipping under statical load
4. Slipping under shock load

The smaller the anchor ∅, the more critical is slipping / pullout.
(small, unfavourable ratio anchor ∅/crack width).

Special Applications: Nuclear Power Plants - Impact

DIBt Guideline for the use of anchors in Nuclear Power Plants (NPP)

Impact categories:

A) probability of 1 occurrence during service life (design earthquake, plane crash, explosions...)

B) probability of ≤10 occurrences during service life

C) service loads (static) and incidents with a probability of ≥10 occurrences during service life

For safety relevant fixings in NPP, DIBt asks for additional tests beside the statical DIBt or ETA-approval.

DIBt: Deutsches Institut für Bautechnik (German Institute for construction technology)

Special Applications: Nuclear Power Plants - Design

DIBt Guideline for the use of anchors in Nuclear Power Plants (NPP)

Design according to concrete-capacity-method, with special load and material safety factors

Load safety factors:

$\gamma_G=\gamma_Q=$ 1.0 for category A
1.2 for category B
1.4 for category C

Material safety factors:

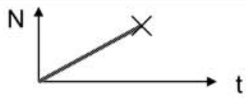
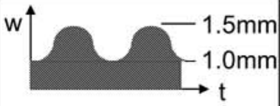
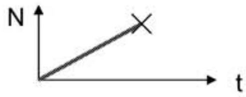
concrete $\gamma_{Mc}=\gamma_{Mp}=$ 1.7 for category A
1.9 for category B
2.1 for category C

steel: according to ETAG, Annex C
(γ_{Ms}=1.4 for tensile loads and γ_{Ms}=1.25 for shear loads)

Special Applications: Nuclear Power Plants - Tests

DIBt Guideline for the use of anchors in Nuclear Power Plants (NPP)

• all anchors are set in closed hairline cracks

	crack width at test	load	criteria
1	1.5 mm		- displacement at 0.5 $F_{u,m}$ - variation of $F_{u,m}$ - value of $F_{u,m}$
2	1.5 mm		- no failure - development of displacement in time
3			- no failure - development of displacement in time
4	1.0 mm		- variation of displacement at $F_{u,m}$ - variation of $F_{u,m}$
5	1.0 mm		- no failure - final ultimate shear load

Hilti BA Fastening & Protection
Anchor Design for Dynamic Loads - part 2: earthquakes and shock

Summary on Earthquake and Shock

- **Earthquakes**
  - Load is a ground motion
  - Load on anchor depends on sytem stiffness
  - Anchors do not bring ductility
  - Design as stiff system with high safety or
  - Design according to approval regulations

- **Shock**
  - Exceptional load
  - Damage to structure accepted
  - Failure must be prevented

- **Nuclear Power Plants**
  - Load cases defined by occurrence
  - Design safety according to load cases
  - Additional testing for safety relevant fastenings

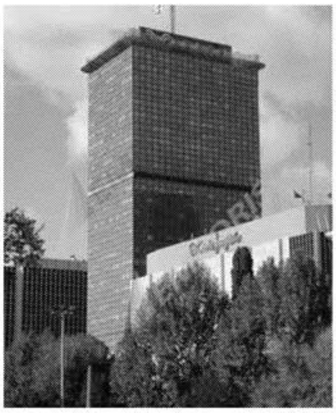
The Madrid Experience

Torre Windsor
Madrid, Spain

Completed : **1979**
Height : **32 storeys (106.0m)**

A landmark tower in Madrid City

11th Feb 2005



The Madrid Experience

The Madrid Experience

12th Feb 2005

.....first flames were sighted on the 21st and 22nd floors of the glass-fronted building. Fire fighters arrived at the scene within minutes...

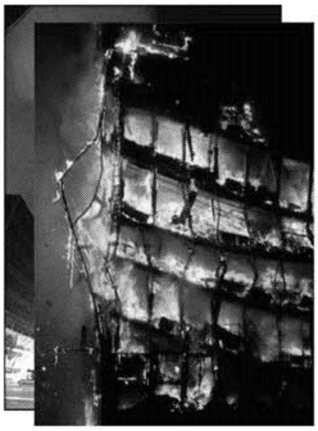
.. although a firecrew that went inside had to withdraw as the blaze spread, engulfing higher floors....

The Madrid Experience

12th Feb 2005

....giant fireballs were seen rising into the night sky as parts of its sides collapsed, raining fire and molten metal onto the streets below.

More than 200 firefighters attempted to bring the blaze under control, pouring water onto the flaming structure from the outside but finding it impossible to reach the upper floors.....



The Madrid Experience

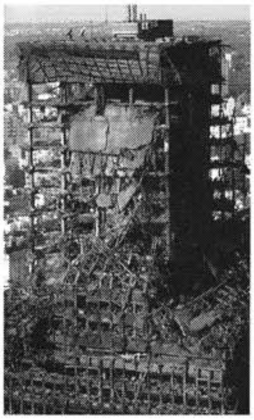
14th Feb 2005

The landmark is no more.

......1/3 of tower and a part of the podium collapsed.

Demolition works will cost 17.5 million euros (S$35m) and will last for aproximately 12 months.

The Madrid Experience



11:05pm	FCC registers fire signal on 21st floor
11:30pm	Fire Fighters start to fight blaze
1:00am	Fire Fighters retreat – too hot + smoke
1:15am	Fire engulfs all floors above 21st, Chunks of façade start to fall, part of tower collapses

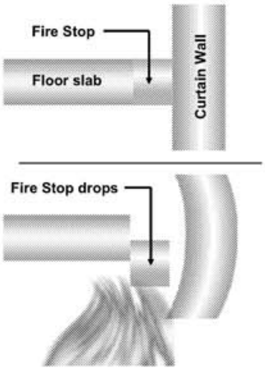
All external slab edge including curtain walls above 17th storey collapsed.

Investigation points to:

- failure of fire compartment system
- lack of sprinkler system
- lack of tie between curtain wall and structure

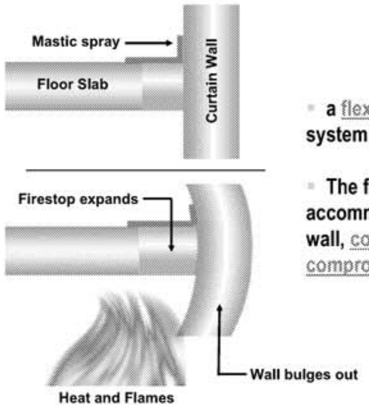
Source : NCE (17th Feb 2005), ARUP Facade (adapted)

The Madrid Experience



- most codes do not address the tie-back connection of the curtain wall to the structure
- light facade structural element can heat up quickly
- expansion of element can produce an outward bulging away from RC slab edge creating internal flues for flame and smoke to travel upwards to the next floor.

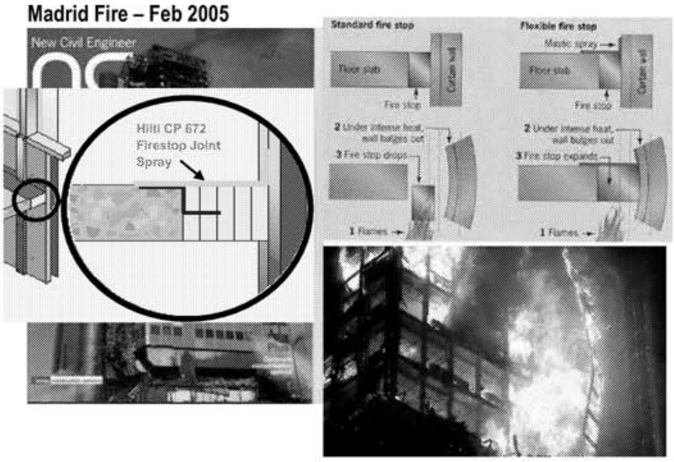
The Madrid Experience



- a flexible expandable firestop system can prevent such problems.
- The firestop system will expand to accommodate the bulging curtain wall, compartmentalisation is not compromised.

Sources : NCE (17th Feb 2005), ARUP Facade (adapted)

Madrid Fire – Feb 2005



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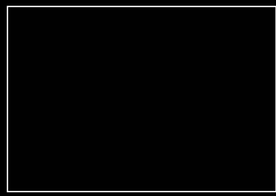
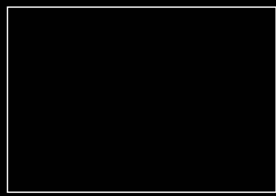
THE END

Think beyond ordinary glass

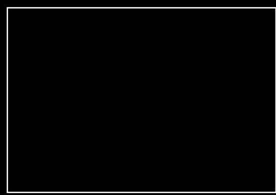
Safety



Security

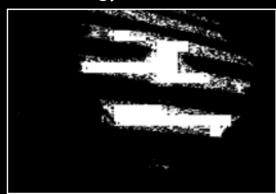


Acoustic Control



Decorative Safety Glass

Solar Energy Control



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