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

Edited by
Mr. H.F. Ren
Mr. Johnny Y.W. Choi
Ir Professor S.L. Chan
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- 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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# Symposium

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

**5th January 2007, Friday**

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

I 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

BD old requirement

APPENDIX A: LOADING TEST

The methods of testing structures or 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 test should be conducted at 0°C.

(10) Retest are. In this case the structure or member should be isolated, in addition to the dead load, to a test load equal to 1.5 times the imposed and wind load, and the loading should be maintained for 24 hours. The maximum deflection specified during this test should not be exceeded. If after removal of the test load, the structure or structure does not show a recovery of at least 90% of the maximum span or deflection shown during the second test, the test should be repeated. The structure should be considered to have sufficient stiffness, provided that the recovery after the second test is not less than 90% of the maximum test or deflection shown during the second test. The deflection of a beam under no design loading should comply with 5.6.

CODE OF PRACTICE FOR THE STRUCTURAL USE OF STEEL 2003

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 some permanent local distortions. The effects 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 an intermediate point of the test load until there is no significant increase in deflection during at least three intervals after the attainment of the test load.
Test load

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). 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 a second test the deflection criteria is reduced to 15% of the maximum recorded during the test.

For a successful strength test the residual deflection on removal of the test load shall not exceed 50% 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/mm² 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 bolt must be sufficiently strong enougth such that the bolt Shank falls 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 D1 below unless test results demonstrate their acceptability in a particular design application.
(d) Friction grip bolts may be tightened using the torque control method, partition method, or direct tension to BS 7684 or other acceptable standard and the manufacturer’s recommendations. Torque spanners and other devices shall be re-calibrated in accordance with BS 4984 or other acceptable standard.

<table>
<thead>
<tr>
<th>Performance requirement</th>
<th>Specified by</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum tensile strength</td>
<td>Tensile testing</td>
</tr>
<tr>
<td>Minimum yield strength</td>
<td>Tensile testing</td>
</tr>
<tr>
<td>Elongation</td>
<td>Tensile testing</td>
</tr>
<tr>
<td>Hardness</td>
<td>Brinell hardness testing</td>
</tr>
</tbody>
</table>

Table D1: Essential performance requirements for bolts

<table>
<thead>
<tr>
<th>Performance requirement</th>
<th>Specified by</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum tensile strength</td>
<td>Tensile testing</td>
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<tr>
<td>Hardness</td>
<td>Brinell hardness testing</td>
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</tbody>
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<table>
<thead>
<tr>
<th>Table D4: Various normally used bolt strengths</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bolt source and grade</td>
</tr>
<tr>
<td>-----------------------</td>
</tr>
<tr>
<td>ISO 4.6</td>
</tr>
<tr>
<td>ISO 8.8</td>
</tr>
<tr>
<td>ISO 10.9</td>
</tr>
<tr>
<td>General grade</td>
</tr>
<tr>
<td>HSFG 2.2</td>
</tr>
<tr>
<td>General grade</td>
</tr>
<tr>
<td>HSFG ≥ 2.7</td>
</tr>
<tr>
<td>High strength</td>
</tr>
<tr>
<td>HSFG</td>
</tr>
<tr>
<td>ASTM A307</td>
</tr>
<tr>
<td>ASTM A325</td>
</tr>
<tr>
<td>ASTM A490</td>
</tr>
</tbody>
</table>

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
Anchored joint
between concrete & steel frame

PNAP 59: Cladding

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
WELDING CONSUMABLES

All welding consumables shall conform to the requirements of the reference standards given in Annex A.1.1. For steel with design strength not exceeding 450 MPa (0 ksi) the specified yield strength, ultimate tensile strength, elongation at failure and Charpy energy values of the welding consumables shall be equal to or better than the corresponding values specified for the grade of steel being welded. The most generous grade shall be used if dissimilar grades are welded together. For high and ultra high strength steels, the welding material may, if necessary, produce a suitable joint, be of a lower strength; the elongation to failure and Charpy impact value shall still match those of the parent material. In that case, the design strength of the weld shall be based on the weld material.
**Kobelco (Japan) – KOBE Steel LB-52**

- 3.2mm diameter & 350mm long
- Standard: AWS A5.1 E7016
- BS EN: 499 E423 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+

---

**MMA electrode constituents and their functions**

<table>
<thead>
<tr>
<th>Constituent</th>
<th>Primary function</th>
<th>Secondary function</th>
</tr>
</thead>
<tbody>
<tr>
<td>Iron oxide</td>
<td>Slag former</td>
<td>Arc stabiliser</td>
</tr>
<tr>
<td>Titanium oxide</td>
<td>Slag former</td>
<td>Arc stabiliser</td>
</tr>
<tr>
<td>Magnesium oxide</td>
<td>Fluxing agent</td>
<td></td>
</tr>
<tr>
<td>Calcium fluoride</td>
<td>Slag former</td>
<td>Fluxing agent</td>
</tr>
<tr>
<td>Potassium silicate</td>
<td>Arc stabiliser</td>
<td></td>
</tr>
<tr>
<td>Other silicates</td>
<td>Slag former and</td>
<td>Fluxing agents</td>
</tr>
<tr>
<td></td>
<td>binders</td>
<td></td>
</tr>
<tr>
<td>Calcium carbonates</td>
<td>Gas former</td>
<td>Arc stabiliser</td>
</tr>
<tr>
<td>Other carbonates</td>
<td>Gas former</td>
<td></td>
</tr>
<tr>
<td>Cellulose</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ferro-manganese</td>
<td>Alloying</td>
<td>Deoxidiser</td>
</tr>
<tr>
<td>Ferro-chrome</td>
<td>Alloying</td>
<td></td>
</tr>
<tr>
<td>Ferro-silicon</td>
<td></td>
<td>Deoxidiser</td>
</tr>
</tbody>
</table>

*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
V-notch Charpy Impact Test

Impact test piece

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
Non-Destructive Tests on Welds

Unaffected parent metal

Weld

Hardness variations in a welded joint in work-hardened material

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 - Assessment of Non-destructive Tests for Welding of steel and Steel

1. Introduction

1.1. This Supplemental Criteria is an explanation and interpretation of the requirements in BS 5135. The path is for the consideration of the subsequent requirements from BS 5135 for welding of steel and steel and the Construction Materials Test Category. The supplement is not to be used for any other purposes.

Volumetric testing (VT)
Liquid Penetrant testing (PT)
Radiographic testing (RT)
Visual Examination (VT)

2. Certification of ASNT shall consist under a quality system complying with ASNT 800. The document gives specific requirements with respect to procurement, measurement, calibration, test methods, testing of welding, repair work, examination, and production testing for work not covered by this document. This document shall be made in accordance with ASNT and ILI. Certificates shall also not test complying with the documents may not necessarily meet the requirements of this standard. Technical services provided may have specific requirements relevant to tests when conducting the tests.
Welded joint
Between reinforcements bars

Welding of Reinforcement Steel Bars

BS 7123

Weldability
Reinforcement may be considered weldable provided the type of steel have the required welding properties given in the acceptance standards. Welding to be inspected and approved by a competent person. Where the weldability is unknown, tests should be carried out.

BD Reinforced Concrete Code

BS 7123

6.7 LAPS AND MECHANICAL COUPLINGS

6.7.1 General
Forces are transmitted from one bar to another by:
- lapping of bars, with or without beads or hooks;
- welding;
- 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 acceptable standards and the recommendations of the reinforcement manufacturer.

Reinforcing bars should not generally be welded at or near bends.
Table 1 — Minimum preheating temperatures for butt joints and cruciform joints: hydrogen controlled consumables

<table>
<thead>
<tr>
<th>Nominal bar size mm</th>
<th>25</th>
<th>&gt; 25 to 40</th>
<th>&gt; 40</th>
</tr>
</thead>
<tbody>
<tr>
<td>Carbon equivalent %</td>
<td>℃</td>
<td>℃</td>
<td>℃</td>
</tr>
<tr>
<td>0.42 or less</td>
<td>0</td>
<td>0</td>
<td>50</td>
</tr>
<tr>
<td>&gt; 0.42 to 0.51</td>
<td>50</td>
<td>75</td>
<td>100</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Nominal bar size mm</th>
<th>25</th>
<th>&gt; 25 to 40</th>
<th>&gt; 40</th>
</tr>
</thead>
<tbody>
<tr>
<td>Carbon equivalent %</td>
<td>℃</td>
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<tr>
<td>0.42 or less</td>
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<td>100</td>
</tr>
<tr>
<td>&gt; 0.42 to 0.51</td>
<td>100</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

BS 7123

13.3 Welding procedure approval tests

13.3.1 General. The following tests shall be carried out:
- 1) full section tensile test;
- 2) macroetch test.

13.3.2 Surface and type of test. 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 7.

13.3.5 Test piece and type of test. 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 7.

Table 6 — Tests required for procedure approval

<table>
<thead>
<tr>
<th>Joint type</th>
<th>Test pieces</th>
<th>Macroetch test</th>
</tr>
</thead>
<tbody>
<tr>
<td>Butt joint</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>Splice joints</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>Lap joints</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>Cruciform joints</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>Test welded lap joint</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Test welded cruciform joint</td>
<td>2</td>
<td>1</td>
</tr>
</tbody>
</table>

13.4 Welder Approval

13.4.1 General

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

13.4.2 Changes not affecting approval

The following changes in the approved welding procedure and the procedure used for the welder approval tests shall not entail reapproval of the welder:

- a change in grade or type of reinforcing bar;
- a change in weld metal composition;
- a change from a basic to a nickel-nickel ferrite;
- a change in preheating temperature.

Table 7 — Tests for welder approval

<table>
<thead>
<tr>
<th>Test piece</th>
<th>Number and type of test</th>
</tr>
</thead>
<tbody>
<tr>
<td>Butt joint</td>
<td>2</td>
</tr>
<tr>
<td>Lap joint</td>
<td>2</td>
</tr>
<tr>
<td>Splice joint</td>
<td>2</td>
</tr>
<tr>
<td>Cruciform joint</td>
<td>2</td>
</tr>
<tr>
<td>Test welded lap joint</td>
<td>1</td>
</tr>
<tr>
<td>Test welded cruciform joint</td>
<td>1</td>
</tr>
</tbody>
</table>

HKIAS Supplementary Criteria No. 3

Construction Product Inspection - Welding Inspection

1.1 This document serves to amplify and interpret the requirements of HKIAS 603 for the accreditation of welding inspection. The welding inspection covered includes inspection/approval of welding processes, inspection/approval of welders, testing, inspection of welding processes and welds quality. The following sections set out specific technical criteria for selected classes of HKIAS 603. For areas not covered in this document, the principles specified in HKIAS 603 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 method 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 specific 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
- HKIAS Approved Signatory for Welding Inspection
- Senior Certified Welding Inspector of AWS
- Senior Welding Inspector of CWSIP
Coupler joint
between reinforcement bars
3.2.8 Mechanical couplers

3.2.8.1 Bars in compression

The load may be transferred between built-in bars by means of end bearing where lower concrete cut ends are held in contact by means of a suitable sleeve or other coupling. 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 built joint between bars in tension is formed using a mechanical coupling satisfying the following criteria.

- When a representative gauge length assembly comprising reinforcement of the diameter, grade and profile to be used, and a coupling of the correct type to be used, is tested in tension the permanent elongation after reaching 0.6% of load not exceed 0.1 mm, and
- the coupled bar assembly tensile strength should exceed 267.5 N/mm² for grade 250, and 490 N/mm² for grade 450.
End of Talk

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

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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 States. 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 °C 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.

![Image of a vaulted roof](image1.jpg)

**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.

![Image of a警察局](image2.jpg)

**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 m² 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).](image)

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 °C, 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.

![Image]

**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](http://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](http://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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Department of Civil and Structural Engineering, The Hong Kong Polytechnic University and

D. Choy and P. Lau

Genetron Engineering Company Limited.

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 ofPreferred ductility not only on bolts, but also on base plate of which the design strength is not allowed to be greater than 270N/mm².

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.

![Beams and Column](image)

**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.
Extended End plate connection

Figure 2 Behaviour of different components at beam column connection

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

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,
\[ EI \frac{d^2 y}{dx^2} + Qx - (F_i + Q)(x - n) = 0 \]  \hspace{1cm} \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, \( \delta_b \), is given by,

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

in which \( Q \) is the prying force shown in Figure 2, \( EI \) is the flexural constant of the connecting plate, \( F_i \) 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

\[ Q = \frac{F_i - 2EI\delta_b/nb^2}{2(n/b) + \frac{2}{3}(n/b)^2} \]  \hspace{1cm} \text{(Eqn 3)}

\( F_i \) 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. \( \delta_b \) is the axial deformation on bolt given in Equation 4.

\[ \delta_b = \frac{(F_i + Q)L_b}{E_b A_b} \]  \hspace{1cm} \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. \( F_i \) is the total tension applied to the connection bolt. Substituting \( \delta_b \) 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,

\[ Q = \frac{F_i \left(1 - \frac{2EIb}{nb^2 E_b A_b}\right)}{2 \left(\frac{n}{b}\right) + \frac{2}{3} \left(\frac{n}{b}\right)^2 + \frac{2EIb}{nb^2 E_b A_b}} \]  \hspace{1cm} \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](image)

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.

$$\frac{F_s}{P_s} + \frac{F_t}{P_{nom}} \leq 1.4$$  \hspace{1cm} (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,

$$\frac{F_s}{P_s} + \frac{F_{tot}}{P_t} \leq 1.4$$  \hspace{1cm} (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](image)

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](image)

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](image)

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

$$P_v = 0.6 \rho_v t \left[ L_v + K_p \left( L_t - kD_t \right) \right]$$  \hspace{1cm} (Eqn 8)

in which $\rho_v$ and $t$ are the design strength and thickness of web of beam or bracket, respectively. $L_v$ and $L_t$ are respectively the length of shear face and tension face shown in Figure 4. $K_p$ 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

Dow Corning Asia
Protective Structural Sealant Glazing

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

Hong Kong
2007 Jan 5th Seminar

Jerry Ma – Dow Corning Asia

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

Only Silicones Have the Properties and Service Life Suited for Structural Glazing

- Silicones adhere glass, transfer windload
- They maintain their physical properties and are unaffected by UV light
- They remain stable, flexible from -40 to 150°C to accommodate thermal expansion
Structural Glazing with DC Silicones – Proven Technology for over 40 years -
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

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)
Severe Weather

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

Falling Glass – A “Life Safety” Issue

- Poor window design or glazing failure may allow glass to spontaneously fall from its frame
Falling Glass – A “Life Safety” Issue

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.

Why use a Silicone?

• Longest lasting sealant available
  – Durability 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
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.

WE HELP YOU INVENT THE FUTURE.™

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

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
Protective Glazing Film System

Triangular joint, most common acceptable

- **Examples**
  
  \[
  A_{\text{min}} = C_{\text{min}} = 6 \text{ mm} \; ; \; B = 6 \text{ mm} \\
  A_{\text{des}} = C_{\text{des}} = 10 - 13 \text{ mm}
  \]

 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 14\textsuperscript{th} 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.

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
Field explosion test

- Parameters:

![Diagram showing reflected pressure and hemispherical surface burst parameters](image)

\[
\begin{align*}
\text{Charge weight, kg} & \quad 12.00 \\
\text{Equivalent weight of TNT, kg} & \quad 12.00 \\
\text{Range, meters} & \quad 6.500 \\
\text{Peak Pressure, kPa} & \quad 392.7 \\
\text{Impulse, kPa-msec} & \quad 547.7 \\
\text{Time of arrival, msec} & \quad 7.361 \\
\text{Duration, msec} & \quad 6.170 \\
\text{Decay coefficient} & \quad 2.111
\end{align*}
\]

---

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
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.

---

Bishops-gate 99 / London


- 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.
DS-2 London

- Architect: Fosters & Partners
- New Design – Canary Wharf
- >15 000 m² 4-sided Structural Glazing with high performance Silicone bonding
- Bomb blast mitigating window design on podium (first 5 stories)

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

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
Exchange Square
Hong Kong

![Graph showing temperature, precipitation, and typhoon/Tropical Storm Signal levels from 1984 to 2004.]

Wan Chai Government – Glass Fallen Down

![Image of a glass panel on a building with the text: Glass panel dropped from Hong Kong's tallest building after a typhoon last year.]

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.

Wyomend says while developers meet the Hong Kong safety standard for wind-loading, it is unnecessary to go a step further now, as damage is usually covered by insurance. He says most should be done in terms of using greater impact resistant materials, such as laminated glass.

Most tall buildings in Hong Kong are designed to withstand about half a metre for every 350 metres of height – the standard wind-loading parameter. Wyomend and his colleagues provide wind protection from minor breezes (through breezes) and stronger winds (strong winds).

"At different levels, an earthquake of 5.5 or above could have disastrous consequences on buildings not adequately designed or built."

It is not unlikely that a great earthquake will hit New Zealand: "The same thing could just as easily happen in Hong Kong."

"Before and after 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."

Wyomend 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 trying on the side of caution, which is good."

"Even in 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$25,000 billion, several years ago, no even damage to a tiny proportion would be expensive.

"Even in 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$25,000 billion, several years ago, no even damage to a tiny proportion would be expensive.

Even though the centre of two of the strongest earthquakes recorded in Hong Kong was 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 Wyomend: "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 one handful of buildings in Hong Kong are designed to resist earthquakes, which include the Mokhondo and LRA-designed Bank of China, the Cheung Kong Centre and Anz-designed Hopewell Centre, and the ICU Building.

A national level of seismic loading is also built in to some hospitals and railway 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 Chai Anz is aware there is no cause for alarm of an earthquake just around the corner, he says seismic regulations are an issue under review in the
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

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.

WE HELP YOU INVENT THE FUTURE.™
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

WE HELP YOU INVENT THE FUTURE.™
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

We help you invent the future.

International Standard for Protective Glazing System

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

WE HELP YOU INVENT THE FUTURE.™
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


**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.
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

The Dow Corning Support You Need

It’s a dangerous world.
Dow Corning can help you make it a little bit safer.
DESIGN AND FABRICATION OF WELDED CONNECTIONS

Ir Dr W T CHAN

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,

$$\text{Derived design strength } P_y = \frac{Y_s}{\gamma_m} \leq \frac{U_s}{\gamma_m2}$$

where $\gamma_m1$ (=1.0) & $\gamma_m2$ (=1.2) are the recommended minimum partial material factors applied to the yield strength and ultimate tensile strength ($U_s$) 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.

\[
CE = C + \frac{Mn}{6} + \frac{(Cu+Ni)}{15} + \frac{(Cr+Mo+V)}{5}
\]

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**

![Diagram of Weld Types]

(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
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.

$$b_e = t_e + 2r_c + 5T_e,$$

but

$$b_e \leq t_e + 2r_c + 5\left(\frac{T_e^2}{t_p}\right)\left(\frac{P_{ye}}{P_{yp}}\right),$$

in which $t_e$, and $T_e$ 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). $P_{ye}$ and $P_{yp}$ are respectively design strength of rolled I- or H-column or structural members and connected plate.

![Figure 5](image)

**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,

$$b_e = 2t_e + 5T_e,$$

but

$$b_e \leq 2t_e + 5\left(\frac{T_e^2}{t_p}\right)\left(\frac{P_{ye}}{P_{yp}}\right),$$

where $t_e$ and $T_e$ 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, \( P_r \geq F_r \)
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](image)

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.
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>
<thead>
<tr>
<th>Steel grade</th>
<th>35</th>
<th>42</th>
<th>50</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Strength (N/mm²)</td>
<td>Strength (N/mm²)</td>
<td>Strength (N/mm²)</td>
</tr>
<tr>
<td>S275</td>
<td>220</td>
<td>220</td>
<td>220</td>
</tr>
<tr>
<td>S355</td>
<td>220</td>
<td>250</td>
<td>250</td>
</tr>
<tr>
<td>S460</td>
<td>220</td>
<td>250</td>
<td>280</td>
</tr>
</tbody>
</table>

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).
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 \( \theta \) 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 \( F_T = \sqrt{F_{T_x}^2 + F_{T_y}^2} \) as shown in Figure 10 (c). The structural adequacy of the weld can be checked by the conditions as,

\[ P_L \geq F_L \]

\[ P_T \geq F_T \]

\[ \left( \frac{F_L}{P_L} \right)^2 + \left( \frac{F_T}{P_T} \right)^2 \leq 1 \]
Figure 10 Directional approach for capacity of fillet weld

\[ a \] Welds subject to longitudinal shear  \hspace{1cm} \[ b \] Welds subject to transverse shear

\[ c \] Resistant transverse force on weld

\textbf{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.

\[ \text{Solution} \]

\[ a \] Partial penetration weld

\textbf{TENSION CAPACITY OF WELD:}
Effective throat thickness, \( \alpha = 16mm < 0.7s = 0.7 \times 25 = 17.5mm \)

Design strength of weld, \( p_s = 280N/mm^2 \)
Capacity of fillet weld, \( P_t = p_s a = 280 \times 16 = 4.48kN/mm \) (transverse to the weld)
Total resistance of this connection, \( P_r = 4.48 \times 300 = 1,344kN > F_j = 200kN \)

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)

![Diagram of vertical plate element]

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, \( p_w = 440N/mm^2 \) \((T \leq 40mm)\)

![Diagram of horizontal plate element]

Effective area of vertical plate element, \( A_v = 40 \times 300 = 12000mm^2 \)

Tension capacity of vertical plate element, \( P_v = p_v A_v = 440 \times 12000 \)

\[= 5280kN > F_j = 1520kN\]

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 cruciform 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.
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.

![End return diagram]

Figure 12 End return of welds
II. Transitional chamfer of joining plates of different thicknesses

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 :-

\[
BI = \frac{\text{CaO} + \text{CaF}_2 + \text{MgO} + \text{K}_2\text{O} + \text{Na}_2\text{O} + 1/2(\text{MnO} + \text{FeO})}{\text{SiO}_2 + 1/2(\text{Al}_2\text{O}_3 + \text{TiO}_2 + \text{ZrO}_2)}
\]

For flux of Basicity Index \( \leq 1 \), 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>
<thead>
<tr>
<th>Weld type</th>
<th>Single</th>
<th>Double</th>
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<tr>
<td>Square</td>
<td><img src="image1" alt="Image" /></td>
<td><img src="image2" alt="Image" /></td>
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<tr>
<td>Bevel</td>
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<tr>
<td>Vee</td>
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<tr>
<td>J</td>
<td><img src="image7" alt="Image" /></td>
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<tr>
<td>U</td>
<td><img src="image9" alt="Image" /></td>
<td><img src="image10" alt="Image" /></td>
</tr>
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</table>
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
c) British Code – BS 449-2: 1969 (now obsolete)
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

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


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.

<table>
<thead>
<tr>
<th>Title of BS</th>
<th>BS</th>
<th>BS EN</th>
<th>Title of BS EN</th>
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<tr>
<td>Covered carbon and carbon manganese steel electrodes for manual metal-arc welding</td>
<td>BS 639:1986 (Superseded, Withdrawn)</td>
<td>BS EN 499: 1995 (Superseded, Withdrawn)</td>
<td>Welding consumables- Covered electrodes for manual metal arc welding of non-alloy and fine grain steels – Classification</td>
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<tr>
<td></td>
<td></td>
<td>by BS EN ISO 2560: 2005 (Current)</td>
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<tr>
<td>Specification for electrode wires and fluxes for the submerged arc welding of carbon steel and medium-tensile steel</td>
<td>BS 4165: 1984 (Superseded, Withdrawn)</td>
<td>BS EN 756: 2004 (Current)</td>
<td>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</td>
</tr>
<tr>
<td>Specification for carbon and carbon-manganese steel tubular cored welding electrodes</td>
<td>BS 7084: 1989 (Revised, Withdrawn)</td>
<td>BS EN 758: 1997 (Current)</td>
<td>Welding consumables- Tubular cored electrodes for metal arc welding with and without a gas shield of non-alloy and fine grain steels – Classification</td>
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<td>-</td>
<td>-</td>
<td>BS EN 719: 1994 (Current)</td>
<td>Welding coordination – Tasks and responsibilities</td>
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<td>Specification for approval testing of welders when welding procedure approval is not required Part 1: Fusion welding of steel</td>
<td>BS 4872-1: 1982 (Current)</td>
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Comparison of BS and BS EN for DT of welds

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<td>Method for penetrant flaw detection.</td>
<td>BS 6443:1984 (Superseded,Withdrawn)</td>
<td>BS EN 571-1:1997 (Current)</td>
<td>Non-destructive testing – Penetrant testing Part 1: General principles</td>
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<tr>
<td>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</td>
<td>BS 2600-1 &amp; 2:1983 (Superseded,Withdrawn)</td>
<td>BS EN 1435:1997 (Current)</td>
<td>Non-destructive examination of welds. Radiographic examination of welded joints.</td>
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## Overview of various national codes on design of welds

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<thead>
<tr>
<th>Code</th>
<th>Design approach</th>
<th>Fillet weld strength</th>
<th>Intermittent Weld</th>
<th>Weld leg length (mm)</th>
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</thead>
<tbody>
<tr>
<td>LRFD: 1999 (USA)</td>
<td>Limit state approach</td>
<td>Using resistance factor $\phi$ multiplied to specified strength of weld to derive design strength</td>
<td>FW&lt;br&gt;Weld length &gt; 4 x weld size, 38 mm min</td>
<td>FW&lt;br&gt;&lt;6 3&lt;br&gt;6-13 5&lt;br&gt;13-19 6&lt;br&gt;&gt;19 8</td>
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<tr>
<td>AS/NZS 1554.1:2000 (Australia)</td>
<td>Limit state approach</td>
<td>No particular mention</td>
<td>FW&lt;br&gt;Weld length &gt; 4t, 40mm min</td>
<td>FW&lt;br&gt;&lt;3 2/3&lt;br&gt;3-7 3&lt;br&gt;7-10 4&lt;br&gt;10-15 5&lt;br&gt;&gt;15 6</td>
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<tr>
<td>BS 449-2: 1969 (UK) (obsolete)</td>
<td>Permissible stress approach</td>
<td>115 MPa for gr.43&lt;br&gt;160 MPa for gr.50&lt;br&gt;195 MPa for gr.55</td>
<td>BW&lt;br&gt;Weld length &lt; 12t&lt;br&gt;Clear dist b/tn weld &lt; 12t</td>
<td>No particular mention</td>
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<tr>
<td>BS 5950: Part 1: 1990 (UK)</td>
<td>Limit state approach</td>
<td>Electrode class 35 42 50&lt;br&gt;design strength 40/43 215 215&lt;br&gt;50 215 255&lt;br&gt;55 255 275</td>
<td>FW&lt;br&gt;Clear dist b/tn weld&lt;br&gt;&lt;16t for compression&lt;br&gt;&lt;24t for tension&lt;br&gt;in no case &gt; 300mm</td>
<td>Min. throat thk for partial penetration butt weld = 2/3</td>
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<tr>
<td>BS 5950: Part 1: 2000 (UK)</td>
<td>Limit state approach</td>
<td>Electrode class 35 42 50&lt;br&gt;design strength S275 220 220 220&lt;br&gt;S355 220 250 250&lt;br&gt;S460 220 250 280</td>
<td>FW&lt;br&gt;Clear dist b/tn weld&lt;br&gt;&lt;16t for compression&lt;br&gt;&lt;24t for tension&lt;br&gt;in no case &gt; 300mm</td>
<td>Min. throat thk for partial penetration butt weld = 2/3</td>
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<tr>
<td>Code</td>
<td>Design approach</td>
<td>Fillet weld strength</td>
<td>Intermittent Fillet Weld</td>
<td>Weld leg length</td>
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</tr>
<tr>
<td>GB 50017-2003 (China)</td>
<td>Limit state approach</td>
<td>Electrode strength class</td>
<td>FW&lt;br&gt;Weld length &gt;10t, 50 min&lt;br&gt;Clear dist btw weld&lt;br&gt;&lt;15t for compression&lt;br&gt;&lt;30t for tension</td>
<td>FW&lt;br&gt;Leg length=1.5t&lt;br&gt;t&lt;6, leg length &lt;t&lt;br&amp;t&gt;6, leg length &lt;t-(1-2)</td>
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<tr>
<td>EN 1993-1-8:2005 (Europe)</td>
<td>Limit state approach</td>
<td>Vector stress&lt;br&gt;&lt; UIt strength/(βw x γm)&lt;br&gt;σL &lt; 0.9 UIt strength/γm&lt;br&gt;where&lt;br&gt;βw is correlation factor</td>
<td>FW&lt;br&gt;Distance btw eff. Length&lt;br&gt;&lt; 12t for comp / shear&lt;br&gt;&lt; 16t for tension&lt;br&gt;in no case &gt; 200 mm&lt;br&gt;Design resistance enhanced by&lt;br&gt;(e+1)/l, where e is clear distance btw weld and l is weld length</td>
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<td>CoP Steel 1987 (HK)</td>
<td>Permissible stress approach</td>
<td>Strength Grade&lt;br&gt;115 MPa for gr.250&lt;br&gt;160 MPa for gr.350&lt;br&gt;195 MPa for gr.450</td>
<td>FW:&lt;br&gt;Distance btw eff. Length&lt;br&gt;&lt;16t for compression&lt;br&gt;&lt;24t for tension&lt;br&gt;in no case &gt; 300mm</td>
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<td>CoP Steel 2005 (HK)</td>
<td>Limit state approach</td>
<td>Electrode class&lt;br&gt;35 42 50&lt;br&gt;design strength&lt;br&gt;S275 220 220 220&lt;br&gt;S355 220 250 250&lt;br&gt;S460 250 280</td>
<td>FW:&lt;br&gt;Distance btw eff. Length&lt;br&gt;&lt;16t for compression&lt;br&gt;&lt;24t for tension&lt;br&gt;in no case &gt; 300mm</td>
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<td>The Building Law of Japan: 1990 (Japan)</td>
<td>Permissible stress approach</td>
<td>F/(1.5x√3)&lt;br&gt;For temp works, F/√3</td>
<td>No particular mention</td>
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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-0). The best way to capture the M-0 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 deck slabs 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.

![Diagram showing general arrangement of test set-up](image)

**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>
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<tr>
<th>Reference</th>
<th>CJ1</th>
<th>CJ2</th>
<th>CJ3</th>
<th>CJ4</th>
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<th>CJ6</th>
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<td>439</td>
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<td>Rotation capacity (mRad)</td>
<td>35.4</td>
<td>33.5</td>
<td>6.1</td>
<td>37.4</td>
<td>31.7</td>
<td>46.8</td>
<td>30</td>
<td>42.3</td>
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<td>Long. reinf. – yield (kN)</td>
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<td>326</td>
<td>326</td>
<td>326</td>
<td>326</td>
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<td>Long. reinf. – Ultimate (kN)</td>
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<td>&gt;100</td>
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</tr>
<tr>
<td>Max. strain in long. reinf. (µε)</td>
<td>26,000</td>
<td>23,000</td>
<td>2,031</td>
<td>16,000</td>
<td>13,706</td>
<td>26,000</td>
<td>23,000</td>
<td>23,000</td>
</tr>
<tr>
<td>Maximum end slip (mm)</td>
<td>0.34</td>
<td>0.8</td>
<td>5.8</td>
<td>3.5</td>
<td>3.5</td>
<td>0.84</td>
<td>0.4</td>
<td>1.6</td>
</tr>
<tr>
<td>Failure mode</td>
<td>RF</td>
<td>RF</td>
<td>CF &amp; SF</td>
<td>CF</td>
<td>CF</td>
<td>RF</td>
<td>RF</td>
<td>RF</td>
</tr>
</tbody>
</table>

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
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>
<thead>
<tr>
<th>Reference</th>
<th>CJ1</th>
<th>CJ2</th>
<th>CJ3</th>
<th>CJ4</th>
<th>CJ5</th>
<th>CJ6</th>
<th>CJ7</th>
<th>CJ8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test result (kNm)</td>
<td>370</td>
<td>363</td>
<td>250</td>
<td>368</td>
<td>363</td>
<td>425</td>
<td>274</td>
<td>439</td>
</tr>
<tr>
<td>FE Model result (kNm)</td>
<td>407</td>
<td>402.9</td>
<td>253.7</td>
<td>383</td>
<td>398</td>
<td>437.6</td>
<td>292</td>
<td>475</td>
</tr>
</tbody>
</table>
### TABLE 4 COMPARISON OF ROTATION CAPACITY

<table>
<thead>
<tr>
<th>Reference</th>
<th>CJ1</th>
<th>CJ2</th>
<th>CJ3</th>
<th>CJ4</th>
<th>CJ5</th>
<th>CJ6</th>
<th>CJ7</th>
<th>CJ8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test result (kNm)</td>
<td>35.4</td>
<td>33.5</td>
<td>6.1</td>
<td>37.4</td>
<td>31.7</td>
<td>46.8</td>
<td>30</td>
<td>42.3</td>
</tr>
<tr>
<td>Modelling result (kNm)</td>
<td>38.5</td>
<td>33.9</td>
<td>11.5</td>
<td>36</td>
<td>36.1</td>
<td>51.4</td>
<td>31.5</td>
<td>49.7</td>
</tr>
</tbody>
</table>

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

**Figure 5** Comparison of Test CJ1 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 assess 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**

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

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.
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, \( M_{c,Rd} \)

\[
M_{c,Rd} = R_r (D_b + D_r - 0.5t_f) + R_b (D_b - r_t - 0.5t_f)
\]  

(1)

where,
- \( D_b \) = the depth of the beam;
- \( r_t \) = 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, $y_c = \frac{(R_x + R_y - R_f)}{t_w p_y}$

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

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

$$M_{c,kd} = R_x(D_y + D_c - 0.5t_l') + R_y(D_y - r_l - 0.5t_l') - R_w \frac{y_c}{2}$$  (2)

where,
$R_w = y_l' t_w p_y$

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>
<thead>
<tr>
<th>Reference</th>
<th>CJ1</th>
<th>CJ2</th>
<th>CJ3</th>
<th>CJ4</th>
<th>CJ5</th>
<th>CJ6</th>
<th>CJ7</th>
<th>CJ8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test result (kNm)</td>
<td>370</td>
<td>363</td>
<td>250</td>
<td>368</td>
<td>363</td>
<td>425</td>
<td>274</td>
<td>439</td>
</tr>
<tr>
<td>Analytical model (kNm)</td>
<td>365.8</td>
<td>365.8</td>
<td>284.5</td>
<td>365.0</td>
<td>366.6</td>
<td>422.3</td>
<td>274.0</td>
<td>446.7</td>
</tr>
</tbody>
</table>

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:

$$\phi_j = \frac{\Delta_y}{D_y + D_c} + \frac{S}{D_y}$$  (3)

In order to determine the elongation of the longitudinal steel bar, the effective deformation length of the longitudinal rebar, $\Delta L$ 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 $P_0 + P_1 + D_r / 2$ 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](image)

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, $\varepsilon_{smu}$ 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**, $\varepsilon_{smu}$

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

(4)

where,

- $\varepsilon_{su}$ is the ultimate strain of the reinforcement
- $\varepsilon_{sy}$ is the yield strain of the reinforcement

The **transmission length**, $L_t$

$$
L_t = \frac{k_c f_{cm} \phi}{4 r_m \rho}
$$

(5)

where,

- $f_{cm}$ is the tensile strength of concrete
- $\rho$ 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 \( k_c = \frac{1}{1 + \frac{h_{es}}{2z_0}} \)

\( h_{es} \) is the thickness of the precast slab

\( z_0 \) 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, \( E_d/E_{cm} \).

\( \phi \) is the diameter of the rebars

\( \tau_{im} \) is the average bond stress along the transmission length and is taken as 1.8 \( f_{cm} \)

For full shear connection, the formula for calculating the elongation of the longitudinal rebar, \( \Delta_r \) is defined as follows:

For \( \rho \leq 1.0 \% \),

\[
\Delta_r = \left( \frac{D_r}{2} + 2L_r \right) \times \varepsilon_{emu}
\]  \hspace{1cm} (6)

For \( \rho > 1.0 \% \) and \( P_0 \leq L_t \)

\[
\Delta_r = \left( \frac{D_r}{2} + 2L_r \right) \times \varepsilon_{emu} + \left( P_1 - L_r \right) \times \varepsilon_y
\] \hspace{1cm} (7)

For \( \rho > 1.0 \% \) and \( P_0 > L_t \)

\[
\Delta_r = \left( \frac{D_r}{2} + P_0 + L_t \right) \times \varepsilon_{emu} + \left( P_1 - L_t \right) \times \varepsilon_y
\] \hspace{1cm} (8)

For partial shear connection, the ultimate mean strain, \( \varepsilon_{emu} \) is taken at the on set of strain hardening if yielding of the longitudinal reinforcement can be achieved. Otherwise, \( \varepsilon_{emu} \) is taken as the yield strain, \( \varepsilon_y \). 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

\[ F_s = \frac{A_s f_y}{n} \]  \hspace{1cm} (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>
<thead>
<tr>
<th>Reference</th>
<th>CJ1</th>
<th>CJ2</th>
<th>CJ3</th>
<th>CJ4</th>
<th>CJ5</th>
<th>CJ6</th>
<th>CJ7</th>
<th>CJ8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test result (mRad)</td>
<td>35.4</td>
<td>33.5</td>
<td>6.1</td>
<td>37.4</td>
<td>31.7</td>
<td>46.8</td>
<td>30</td>
<td>42.3</td>
</tr>
<tr>
<td>Analytical method (mRad)</td>
<td>29.1</td>
<td>31.3</td>
<td>18.7</td>
<td>30.0</td>
<td>28.7</td>
<td>43.4</td>
<td>27.9</td>
<td>53.6</td>
</tr>
</tbody>
</table>
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.
REFERENCES


ANCHOR DESIGN FOR DYNAMIC LOADS – EARTHQUAKES AND SHOCK

Dr J. Kunz

Hilti (Hong Kong) Ltd
Basics: Possible Dynamic Actions

characteristic of dynamic actions:
induced accelerations activate forces of inertia and damping

<table>
<thead>
<tr>
<th>Classification</th>
<th>Fatigue</th>
<th>Seismic</th>
<th>Shock</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frequency of appearance</td>
<td>$10^4 &lt; n &lt; 10^8$</td>
<td>$10^1 &lt; n &lt; 10^4$</td>
<td>$1 &lt; n &lt; 20$</td>
</tr>
<tr>
<td>Rate of strain</td>
<td>$10^{-6} &lt; \varepsilon' &lt; 10^{-3}$</td>
<td>$10^{-5} &lt; \varepsilon' &lt; 10^{-2}$</td>
<td>$10^{-3} &lt; \varepsilon' &lt; 10^{-1}$</td>
</tr>
</tbody>
</table>
| Examples               | • Traffic loads  
                        • Machines, cranes  
                        • Ventilators  
                        • Wind, Waves | • Earthquakes  
                        • Artificial earthquakes | • Explosions  
                        • Abrupt Structure failure  
                        • Crash Barriers |
Anchor Design for Dynamic Loads - part 1: basic principles and fatigue

**Basics: Possible Dynamic Actions**

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

**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 Challenges

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 Ø → 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

[Graph showing Wöhler Curves with various load cycles and amplitudes]
Fatigue: 2. Determination of Fatigue Relevant Loads

\[ \Delta F = \sqrt{\Delta N^2 + \Delta V^2} \]


Different anchors in anchor group have different stiffness, due to:
- verticality
- cracks

Load redistribution: stiffer anchors are higher loaded and therefore will fail earlier.

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

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.

Dynamic Set

For a perfect fatigue fastening the "Dynamic Set" has to be used.
Fatigue: Design

Safety Concept According to Eurocode

dynamic action
characteristic resistance under dynamic action
partial safety factor (resistance)
design value of resistance under dynamic action
design

design value of dynamic action
partial safety factor (action)
characteristic value of dynamic action

\[ R_d = R_k / \gamma_F \]
\[ S_d = S_k \cdot \gamma_F \]

\[ R_{d,\text{fatigue}} \quad \gamma_F,\text{fatigue} \]
\[ R_{d,\text{shock}} \quad \gamma_F,\text{shock} \]
\[ R_{d,\text{earthquake}} \quad \gamma_F,\text{earthquake} \]

\[ S_{d,\text{fatigue}} \quad \gamma_F,\text{fatigue} \]
\[ S_{d,\text{shock}} \quad \gamma_F,\text{shock} \]
\[ S_{d,\text{earthquake}} \quad \gamma_F,\text{earthquake} \]

Fatigue: Design for Anchors - Hilti method

Static check

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

Tension
\[ \min(N_{Rd,e}, N_{Rd,s}) \geq N_d + \Delta N_d \]

Shear
\[ \min(V_{Rd,e}, V_{Rd,s}) \geq V_d + \Delta V_d \]

Combined load
\[ F_{sd}(\alpha) = F_{rd} = \left( \cos \alpha \frac{1}{N_{Rd}} \right)^{1.5} + \left( \sin \alpha \frac{1}{V_{Rd}} \right)^{2.3} \]
Fatigue: Design for Anchors - Hilti method

**Pretension influence**

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

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

[Diagram showing load and displacement over time with equations and variables]

**Friction** is characterised by:

- Friction factor $\mu$
- Prestressing force $F_V$

Friction force $F_R = F_V \times \mu$

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 F₁ and fatigue load ΔF₁ (steel failure)
2b Combination of static load F₂ and fatigue load ΔF₂ (concrete failure)

Fatigue check
\[ \frac{\Delta N_{sd}}{\Delta N_{rd}} \leq 1.0 \]
Fatigue: Design for Anchors - Hilti method

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

resistance
a) Wöhler-charts

\[ \Delta N_{Rd,c} \leq \Delta N_d \]
\[ \Delta V_{Rd,c} \leq \Delta V_d \]

b) ln. Weyrauch-charts

\[ \Delta N_{Rd,c} \leq \Delta N_d \]
\[ \Delta V_{Rd,c} \leq \Delta V_d \]

fatigue check
\[ \Delta N_{Rd,c} \leq \Delta N_d \]
\[ \Delta V_{Rd,c} \leq \Delta V_d \]

static check
\[ \min (N_{Rd,c}, N_{Rd,sa}) \geq N_d + \Delta N_d \]
\[ \min (V_{Rd,c}, V_{Rd,sa}) \geq V_d + \Delta V_d \]

friction check
\[ \mu : N_{Rd,min} \geq V_d' + \Delta V_d' \]

pretension

Acting loads
- static: \( N_d, V_d \)
- alternating: \( \Delta N_d, \Delta V_d \)

friction: \( \mu \)

nbr. cycles: \( n \)

Fatigue: Design for Anchors - DIBt1) - method

Simplified Assumptions

- determination of fatigue loads (all loads are fatigue relevant)
- for all number of load cycles \( n \geq 10'000 \Rightarrow n=2'000'000 \)
- load safety factor \( \gamma_F=1.0 \)
- 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

<table>
<thead>
<tr>
<th>Static Design (cc-method)</th>
<th>Dynamic Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load safety factors</td>
<td>Load safety factors $\gamma_F = 1.0$</td>
</tr>
<tr>
<td>$\gamma_C = 1.35$ acc. Eurocode</td>
<td>Group factors $\gamma_{F,N}$, $\gamma_{F,V}$</td>
</tr>
<tr>
<td>$\gamma_Q = 1.5$ acc. Eurocode</td>
<td>$= 1.0$ for single anchors; $&gt; 1.0$ for group fastenings</td>
</tr>
<tr>
<td>Characteristic resistances related to static concrete and steel strength</td>
<td>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</td>
</tr>
<tr>
<td>$N_{Rk,s}$, $N_{Rk,c}$, $V_{Rk,s}$, $V_{Rk,c}$</td>
<td>Different material safety factors $\gamma_{M,s}$, $\gamma_{M,c}$</td>
</tr>
</tbody>
</table>

Fatigue check

$tension, concrete$ $\Delta N_{Rd,s} > \Delta N_d$

$tension, steel$ $\Delta N_{Rd,s} > \Delta N_d$

$shear, concrete$ $\Delta V_{Rd,c} > \Delta V_d$

$shear, steel$ $\Delta V_{Rd,c} > \Delta V_d$

Static check

Friction check

Pretension

load

Static: $N^s_d$, changing: $\Delta N^s_d$, $V^s_d$, $\Delta V^s_d$

Friction: $\mu$, nbr. cycles: $n$
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
Earthquake
Impact on Fasteners
Cracks, Ductility
Testing, Research

Shock
Impact on Fasteners
Testing
Resistance of Anchors

Special Application: Nuclear Power Plants

Earthquakes: Risk of Earthquakes

Ring of Fire
Earthquakes: Examples
Typical Fastening Problems (structural and non structural)

- Suspended Ceilings
  - San Francisco Airport
  - Loma Prieta earthquake 1989

- Damages in a Switchyard
  - Managua, 1972

- Toppled and damaged furniture
  - City Hall Kobe, 1995

- Falling wall panels
  - Public Building, Kobe, 1995

- Destroyed column fixing
  - San Francisco, 1989

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

- 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

Earthquakes: Design

Example: Uniform Building Code (USA)

- estimation of horizontal load (static)

\[ F_p = Z \cdot I \cdot C_p \cdot W_p \]

- \( F_p \): 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

"fastenlers shall be designed for \textbf{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

![Diagram showing load vs. cycles with values $N_s$, $N_t$, $N_m$. $N_s$ is the maximum tension test load, $N_t$ is a load midway between $N_s$ and $N_m$, and $N_m$ is 1/4 of the average ultimate tension load $T_{ref}$.]
Earthquakes: Testing of Anchors - Shear

- Maximum load $V_s$: 1.5 times the 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

![Diagram showing load vs. cycles](image)

- $V_s$: 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 $T_{ref}$

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Research for Earthquake Retrofitting

![Image of a room with retrofitted structures](image)
Anchor Design for Dynamic Loads - part 2: earthquakes and shock

Research for Earthquake Retrofitting

experimental setup at Shibaura Institute of Technology Tokyo

"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: \( P_{RK} = 0.8 f_u \cdot \frac{\pi \cdot d^2}{4} \)

concrete bearing pressure: \( P_{RK} = 0.29 d^2 \sqrt{f_{ck} \cdot E_c} \)
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.

Shock: Examples

Typical Fastening Problems (structural)

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

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

- Explosions (e.g. installations in civil shelters)
Shock: Testing

- single anchor in cracked concrete (w=1.0mm!) - concrete quality ~C20/25
- Load: 1.5*N_{rec} for “older” anchors / N_{a} 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 \( \varnothing \), the more critical is slipping / pullout.
(small, unfavourable ratio anchor \( \varnothing \)/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
statitical DIBt or ETA-approval.

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

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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:

\[ \gamma_0 = \gamma_0^* = \begin{cases} 
1.0 & \text{for category A} \\
1.2 & \text{for category B} \\
1.4 & \text{for category C} 
\end{cases} \]

Material safety factors:

- concrete: \( \gamma_{M_0} = \gamma_{M_p} \) = \begin{cases} 
1.7 & \text{for category A} \\
1.9 & \text{for category B} \\
2.1 & \text{for category C} 
\end{cases} \)

- steel: according to ETAG, Annex C
  \[ \gamma_{M_s} = 1.4 \text{ for tensile loads and } \gamma_{M_s} = 1.25 \text{ for shear loads} \]
### 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

<table>
<thead>
<tr>
<th>crack width at test</th>
<th>load</th>
<th>criteria</th>
</tr>
</thead>
</table>
| 1                   | ![Graph](image1) | - displacement at 0.5 $F_{u,m}$  
                       |      | - variation of $F_{u,m}$  
                       |      | - value of $F_{u,m}$ |
| 2                   | ![Graph](image2) | - no failure  
                       |      | - development of displacement in time |
| 3                   | ![Graph](image3) | - no failure  
                       |      | - development of displacement in time |
| 4                   | ![Graph](image4) | - variation of displacement at $F_{u,m}$  
                       |      | - variation of $F_{u,m}$ |
| 5                   | ![Graph](image5) | - no failure  
                       |      | - final ultimate shear load |

### Summary on Earthquake and Shock

- **Earthquakes**
  - Load is a ground motion  
  - Load on anchor depends on system 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

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

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 (5835m) and will last for approximately 12 months.

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

11:05pm  FCC registers fire signal on 21st floor
11:10pm  Fire fighters start to fight blaze
1:15am  Fire engulfs all floors above 21st.
        Chunks of facade 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: ICE (11th 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.

Madrid Fire – Feb 2005

THE END
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Proven and approved in cracked concrete