



# 國家運輸安全調查委員會

## 重大運輸事故調查報告 第二冊

中華民國 108 年 10 月 1 日

南方澳大橋斷裂重大公路事故

報告編號：TTSB-HOR-20-11-001

報告日期：民國 109 年 11 月

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附錄 1 吊索勘查紀錄



事故調查技術報告

南方澳大橋斷裂重大運輸事故

吊索勘查紀錄

報告編號：TTSB-TFR-20-04-001R01

報告日期：民國 109 年 04 月

技術報告/公路/工程	國家運輸安全調查委員會 運輸工程組
專案：吊索勘查紀錄	
日期：108 年 10 月 5 日 108 年 10 月 16 日 108 年 11 月 28 日	執行人員：
地點：橋面、蘇澳港 8 號碼頭、蘇澳港 11 號倉庫	
版本：R01 增加文字描述	

## 1. 概述

本事故雙叉式單拱型橋梁之橋面承載採用吊索支撐設計，即承受所有來自橋面方向的張力負載，並傳遞至拱架，全橋 13 具吊索的破壞狀況可直接反映破壞起始點，以及失效次序推估最重要的資訊之一。

## 2. 吊索破壞狀況

依據本會先遣小組空中測繪資料及國震中心提供 LiDAR 掃描成果，第 2、3、4、6 及部分第 5、7、8、9、10 束吊索脫離或斷裂後完全浸泡於海水中，詳圖 1 至圖 7。

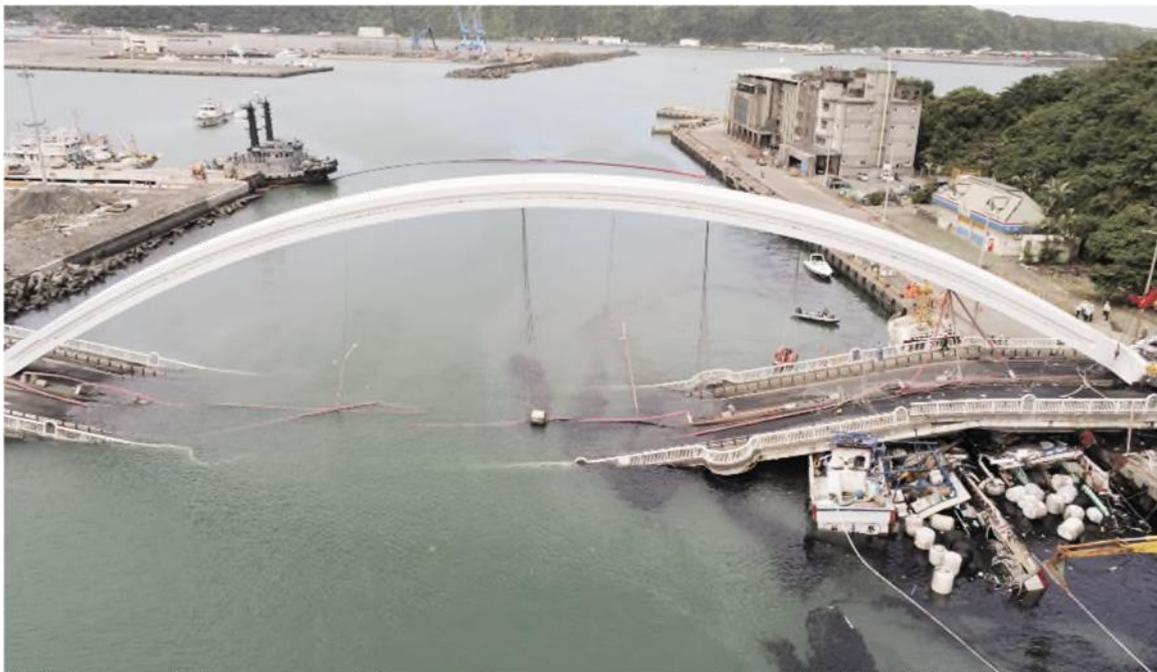


圖 1、事故後吊索狀況

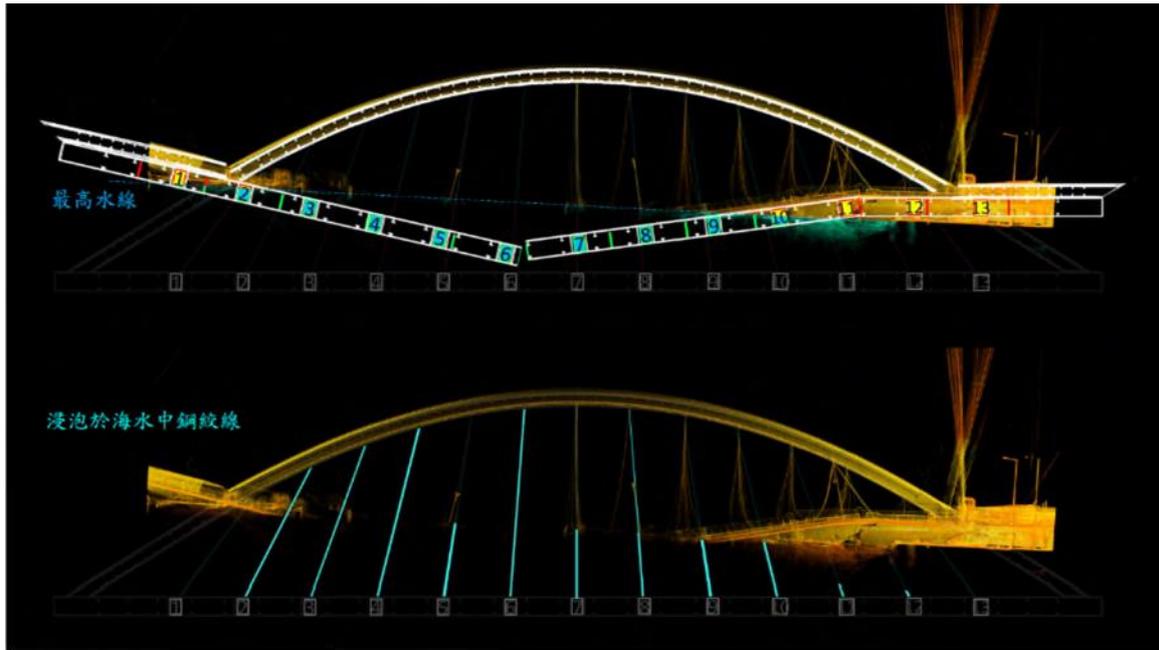


圖 2、浸水吊索（藍色處）與水線套疊示意圖



圖 3、第 1-9 號吊索入水狀況（面北）



圖 4、第 1-7 號吊索入水狀況 (面南)

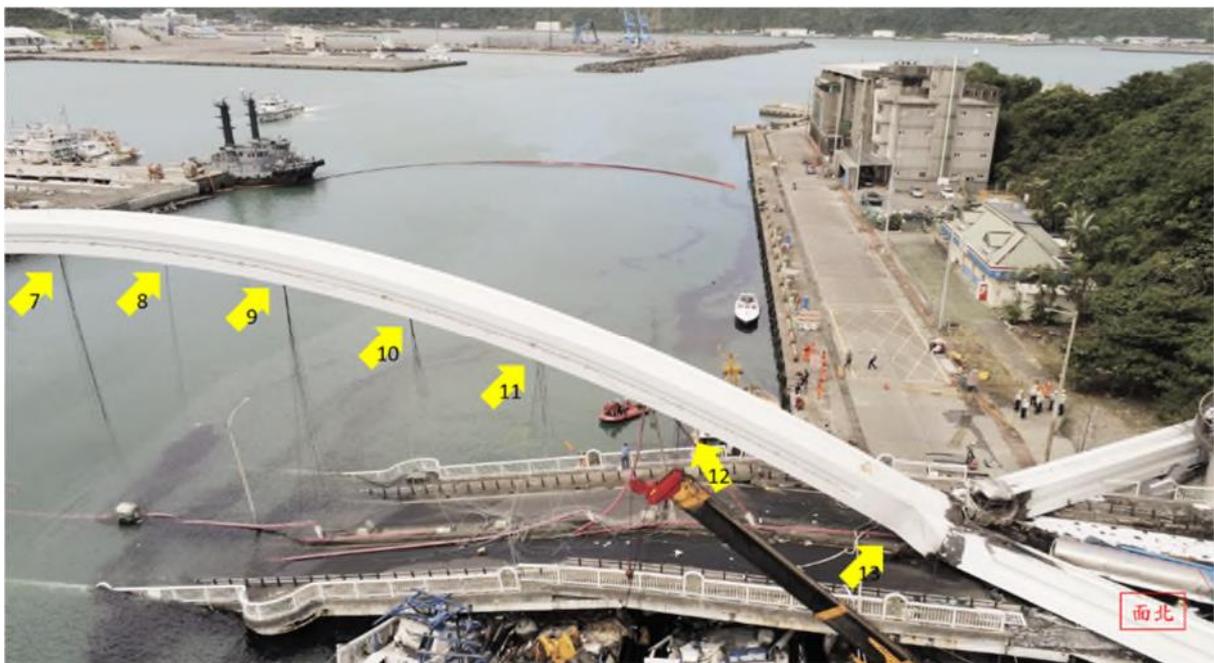


圖 5、第 7-13 號吊索入水狀況 (面北)

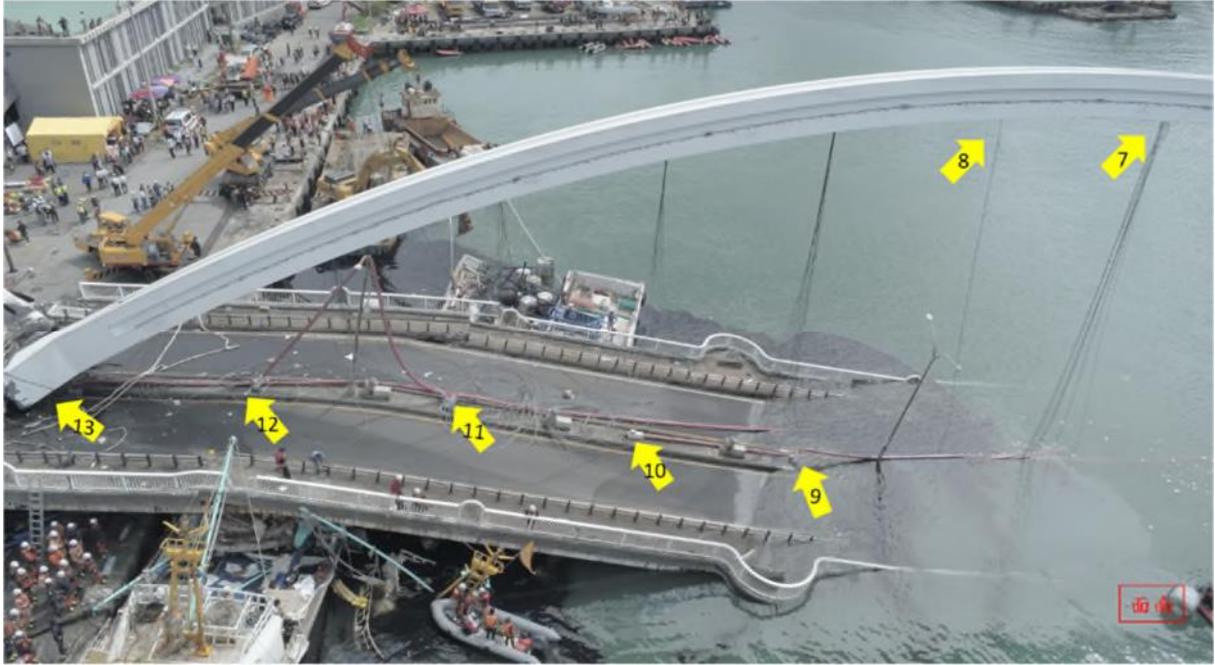


圖 6、第 9-13 號吊索入水狀況（面南）

鋼絞線甚長且重，事故地點臨海且部分鋼絞線沈於海水中，不利於保存金屬材質證物，故依施工安全性及運送存放限制，進行分段裁切或捆捲後，移運至蘇澳港 11 號倉庫存放，以遂後續勘查蒐證。

第 1 至 13 號吊索內共 177 束鋼絞線，除少量線段因於海上切除時落入海中未收回外，大橋所使用鋼絞線總長度超過約 90% 均依序陳列於存放場域內。調查小組對所有回收之鋼絞線進行受損狀況分類、統計、分布等進行資料彙整，部分較輕微或細部之破壞型態發現有如下：

- 末段退穿 (extruded)
- 心線位移 (displaced)
- 心線遺落 (remaining)
- 心線遺失 (missing)
- 鋼線脫離凸出 (protruded)

其餘因大橋斷裂時所受損之鋼絞線分類及分布如圖 7 至圖 12，主要較顯著<sup>1</sup>破壞型態包含：

- 鋼絞線之末端完整有夾片壓痕且與錨頭脫離 (escaped)

<sup>1</sup> 標示之破壞非吊索中所有鋼絞索皆發現遭受相同破壞行為。

- 退絞解離 (detwisted) 或四散 (open-end)
- 頸縮 (necking) 斷線
- 烏籠現象 (caging)
- 鏽蝕 (corrosion)

鋼絞線破壞狀況檢視紀錄詳下表，除上述事故前之破壞型態，事故後工程人員所進行之人為切割或凹折等僅記錄存參不於報告中呈現。

### 2.1 第 1 號吊索：

拱架端：

- 17 束絞線末端呈現輕微退絞現象



<p>橋面端：</p> <ul style="list-style-type: none"><li>● 17 束末端脫離錨頭<ul style="list-style-type: none"><li>◇ 7 束且解離</li><li>◇ 2 束輕微退絞</li><li>◇ 2 束表面有夾片壓痕，末端完整</li><li>◇ 2 束仍帶有破斷錨頭結構碎片</li><li>◇ 2 束無特殊發現</li></ul></li></ul>	
<p>其他：</p> <ul style="list-style-type: none"><li>● 17 束皆未斷</li></ul>	

2.2 第 2 號吊索：

<p>拱架端：</p> <ul style="list-style-type: none"><li>● 13 束末端脫離錨頭且呈現退絞解離現象</li></ul>	
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<p>橋面端：</p> <ul style="list-style-type: none"><li>● 13 束末段退穿 (extruded) 錨頭夾片處，且尾端解離四散</li></ul>	
<p>其他：</p> <ul style="list-style-type: none"><li>● 13 束皆未斷</li></ul>	

2.3 第 3 號吊索：

<p>拱架端：</p> <ul style="list-style-type: none"><li>● 13 束末端脫離錨頭<ul style="list-style-type: none"><li>◇ 2 束表面有夾片壓痕，末端完整</li><li>◇ 8 束退絞解離</li><li>◇ 3 束輕微退絞</li></ul></li></ul>	
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<p>橋面端：</p> <ul style="list-style-type: none"> <li>● 13 束末段退穿出錨頭夾片處</li> <li>◇ 12 束尾端解離四散</li> <li>◇ 1 束脫離錨頭，表面有夾片壓痕，末端完整</li> </ul>	
<p>其他：</p> <ul style="list-style-type: none"> <li>● 13 束皆未斷</li> </ul>	

2.4 第 4 號吊索：

<p>拱架端：</p> <ul style="list-style-type: none"> <li>● 13 束末端脫離錨頭</li> <li>◇ 4 束表面有夾片壓痕，末端完整</li> <li>◇ 2 束末端輕微退絞</li> <li>◇ 7 束末端退絞解離</li> </ul>	
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橋面端：

- 13 束鋼絞線末端皆留於錨頭上：
- ◇ 5 束有心線位移退出現象
- ◇ 5 束輕微退絞
- ◇ 3 束狀況良好
- ◇ 錨頭外之鋼絞線長度不一



其他：

- 13 束皆未斷

## 2.5 第 5 號吊索

拱架端：

- 13 束鋼絞線，12 束末端脫離錨頭，1 束退絞解離仍留於錨頭上，末端輕微退絞解離
  - ◇ 其中 3 束表面有夾片壓痕，末端完整
  - ◇ 其中 1 束末端完整情中兩束仍帶有破斷錨頭結構碎片
  - ◇ 其中 9 束輕微退絞
- 13 束末端表面均呈現鏽蝕現象



	
<p>橋面端：</p> <ul style="list-style-type: none"> <li>● 13 束皆留於錨頭上             <ul style="list-style-type: none"> <li>◇ 7 束接近錨頭處多處鳥籠</li> </ul> </li> <li>● 13 束皆有表面輕微鏽蝕現象</li> </ul>	
<p>其他：</p> <ul style="list-style-type: none"> <li>● 13 束皆未斷</li> </ul>	

## 2.6 第 6 號吊索

拱架端：

- 13 束末端脫離錨頭
  - ◇ 8 束表面有夾片壓痕，末端完整
  - ◇ 3 束尾端輕微退絞
  - ◇ 2 束退絞解離



橋面端：

- 13 束末段退穿出錨頭夾片處且尾端退絞解離四散
- ◇ 1 束末端輕微表面鏽蝕及鳥籠現象



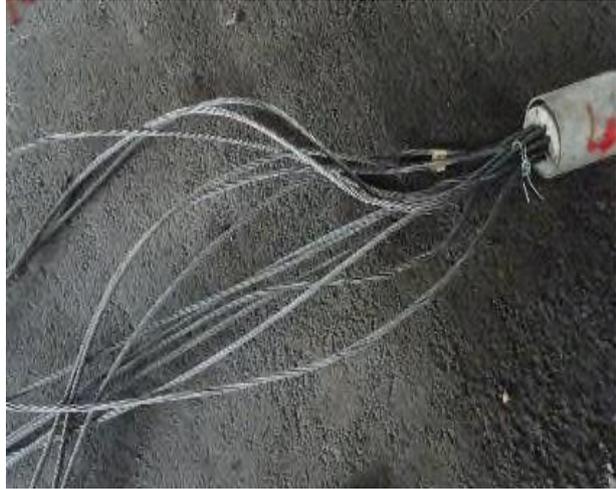
其他：

- 13 束皆未斷裂

## 2.7 第 7 號吊索

拱架端：

- 13 束其中 12 束接近錨頭處退絞輕微烏籠，另一束狀況良好



橋面端：

- 13 束末端退絞解離四散且輕微鏽蝕
  - ◇ 10 束末端脫離錨頭
  - ◇ 3 束有頸縮斷裂現象，位置靠近下錨頭但仍留於錨頭上，其中 2 束有烏籠現象，1 束有心線遺落現象



## 2.8 第 8 號吊索

拱架端：

- 13 束鋼絞線中：
  - ◇ 2 束鋼絞索未斷留於破裂錨頭內
  - ◇ 3 束鋼絞線斷於夾片處線頭留於錨頭內
  - ◇ 8 束位於錨頭中央部分脫離錨頭，部分連帶錨頭破裂碎片之鋼絞線段遺失



橋面端：

- 13 束靠近錨頭處有退絞情形
  - ◇ 其中 2 束烏籠現象較為明顯



其他：

- 第 8 號鋼絞索有 3 束部分落海未取回，大致位於前段與中間段，總長約 20 公尺



## 2.9 第 9 號吊索

拱架端：

- 13 束皆留於錨頭上：
  - ◇ 開蓋後鋼絞索纏繞於外殼內頂端
  - ◇ 多束尾端退絞解離
  - ◇ 帽蓋桶殼內鋼絞索未鏽蝕





橋面端：

- 13 束皆留於錨頭，且：
  - ◇ 有頸縮斷裂現象
  - ◇ 靠近錨頭處退絞解離
  - ◇ 斷裂處嚴重鏽蝕



其他：

- 第九號吊索有 6 束整束均呈現表面鏽蝕現象，另 7 束未鏽蝕



## 2.10 第 10 號吊索

拱架端：

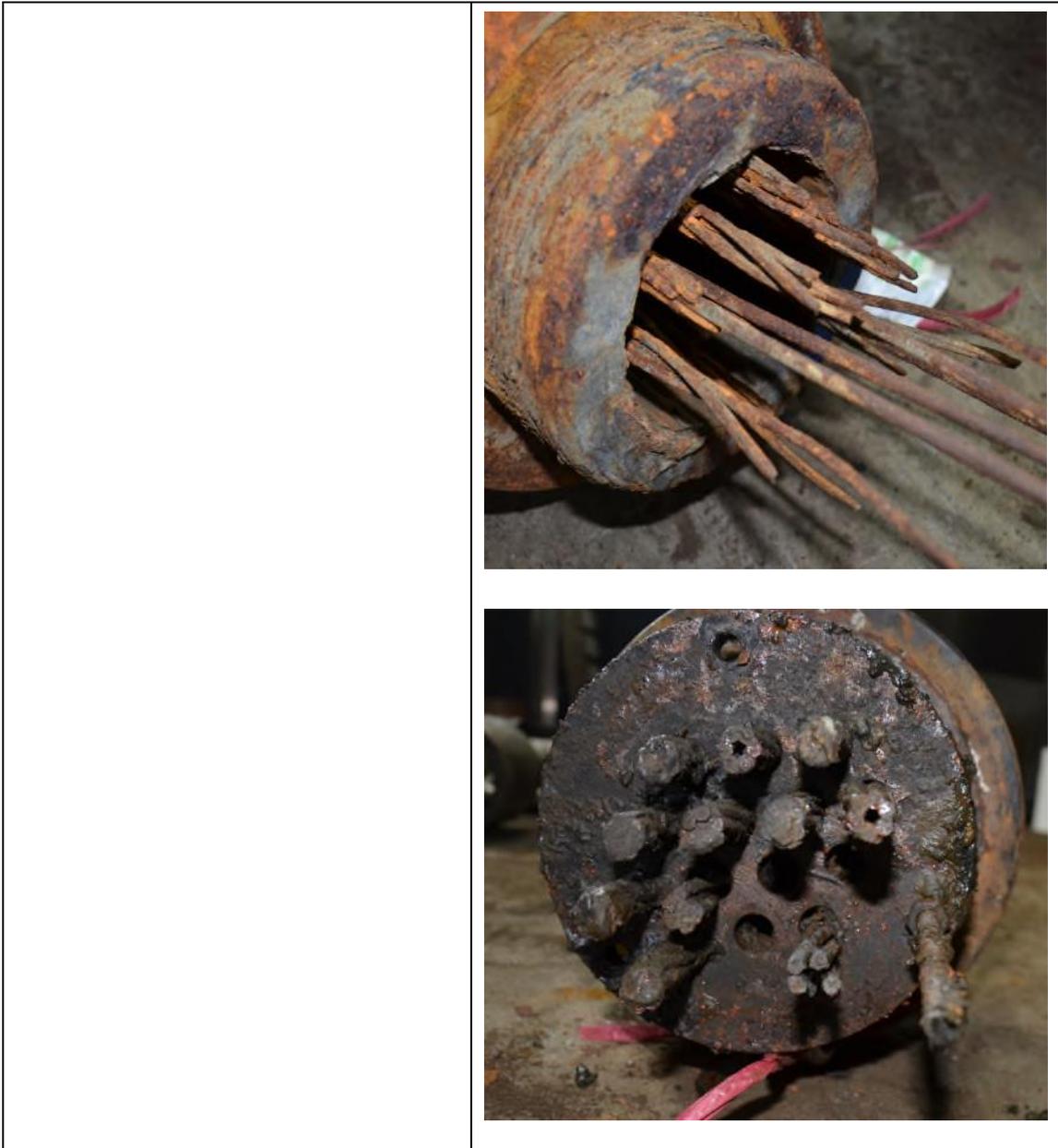
- 13 束仍留於錨頭上，外觀完整



橋面端：

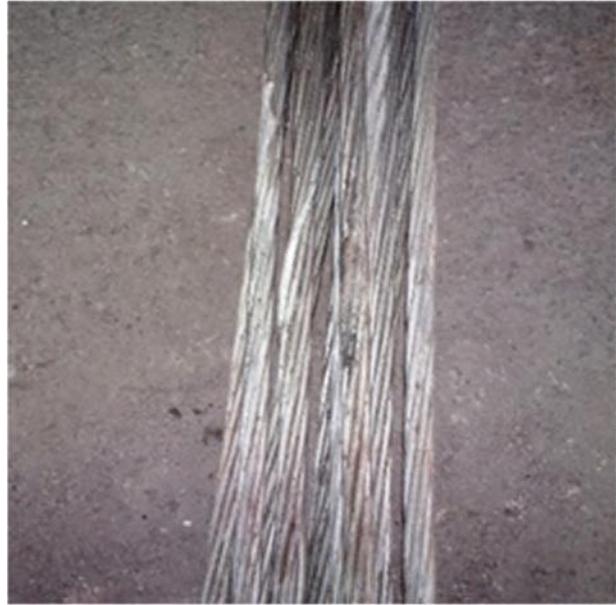
- 13 束鋼絞線中：
  - ◇ 11 束於接近錨頭處斷裂，有頸縮現象，另 2 束未留於錨頭內，斷裂區域嚴重鏽蝕
  - ◇ 其中 4 束有心線遺落退出現象





其他：

- 靠近錨頭處有 2 處輕微退絞

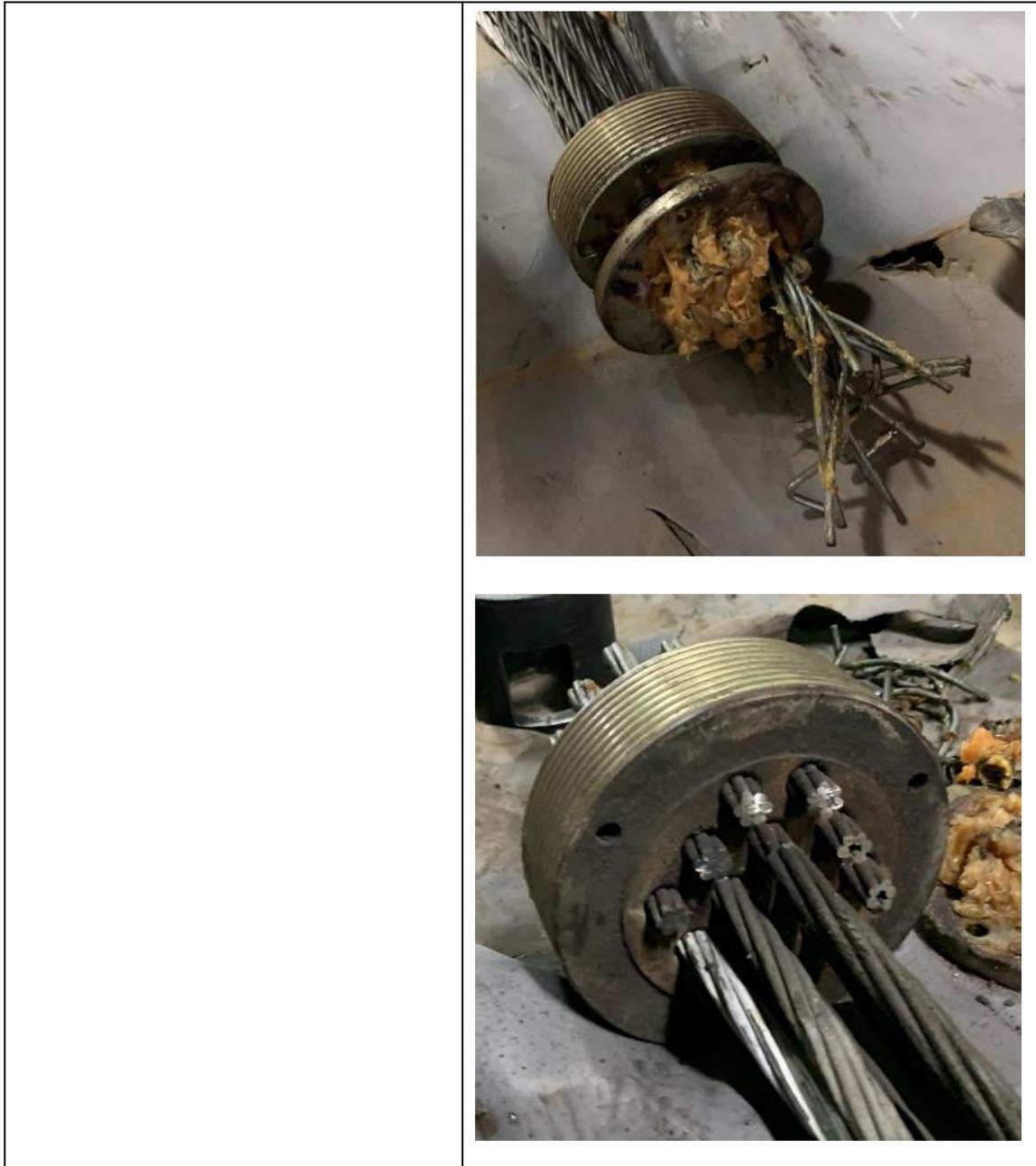


## 2.11 第 11 號吊索

拱架端：

- 13 束皆留於錨頭內：
  - ◇ 7 束有輕微烏籠現象
  - ◇ 4 束靠近錨頭處有部分斷裂現象，斷裂處呈現鏽蝕現象
  - ◇ 2 束無異常發現
- 去除帽蓋後，發現其中 2 束自錨頭退出，退絞頂住帽蓋頂端，4 束錨頭處輕微退絞解離，另 3 束有心線遺失現象







橋面端：

- 13 束鋼絞線中：
  - ◇ 10 束錨頭處有斷絲、頸縮斷裂及退絞解離現象，其中 2 束有心線遺失，1 束心線遺落
  - ◇ 3 束未斷
  - ◇ 接近錨頭位置部分有表面鏽蝕現象



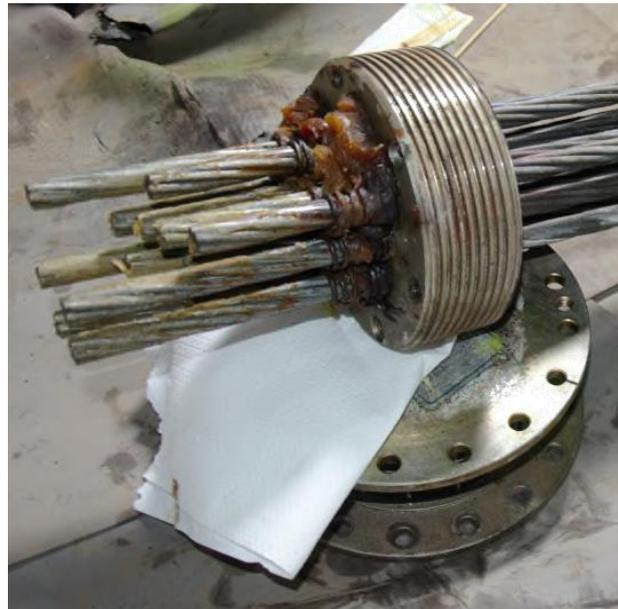




## 2.12 第 12 號吊索

拱架端：

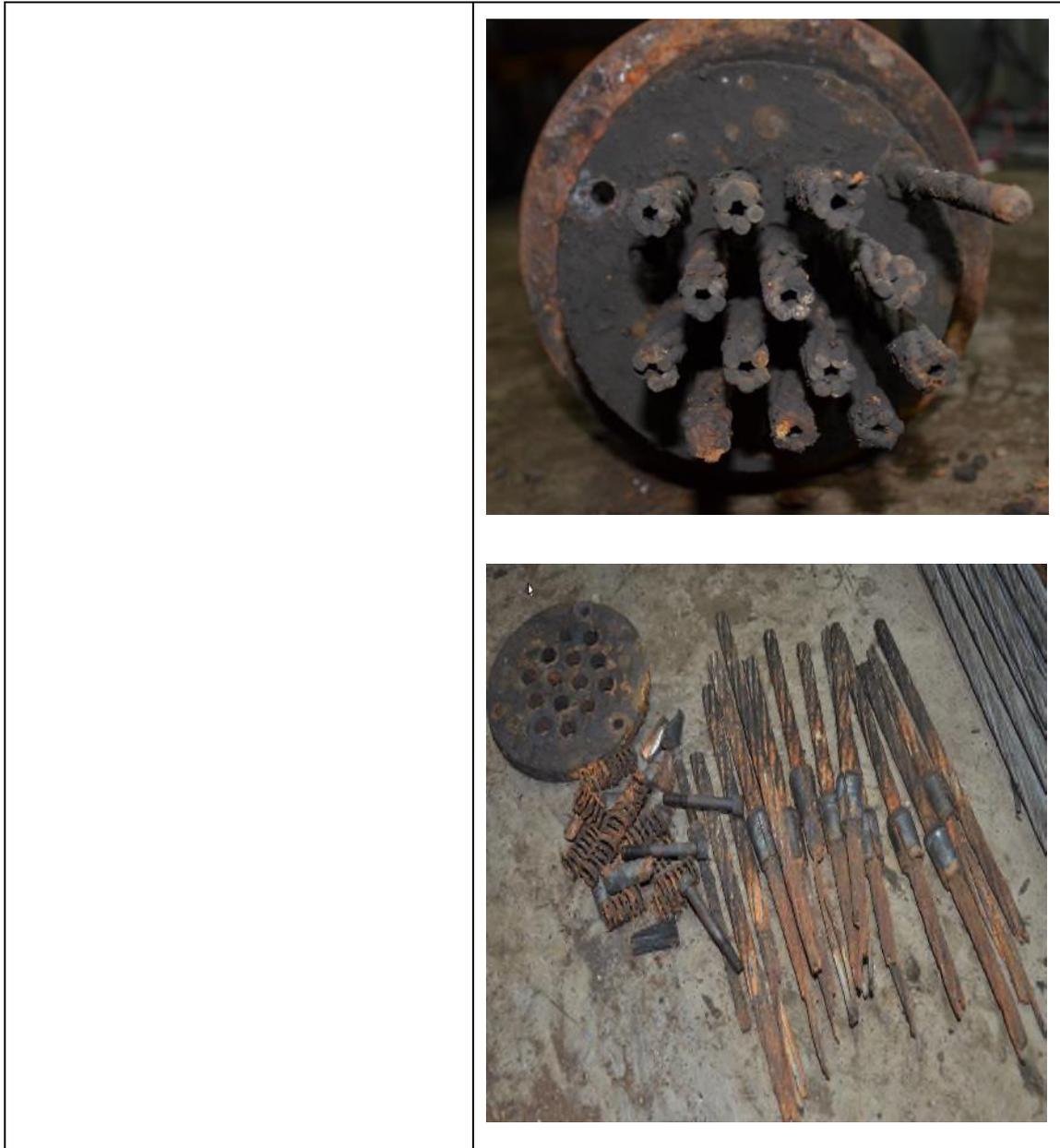
- 13 束末端完整
  - ◇ 其中 3 束於近錨頭處發生輕微烏籠現象



橋面端：

- 13 束錨頭處鋼絞索頸縮  
斷裂現象
  - ◇ 10 束心線遺失退離
  - ◇ 斷於錨頭內殘線嚴重  
鏽蝕





2.13 第 13 號吊索

拱架端：

- 17 束鋼絞線中：
  - ◇ 8 束末端脫離錨頭
  - ◇ 9 束末段退穿出錨頭處約 2 公尺，尾端解離四散





橋面端：

- 17 束皆斷於靠近錨頭夾片處
  - ◇ 部分鋼絞線斷面未完全鏽蝕，核心處仍呈現金屬光澤
  - ◇ 9 束末端解離四散且有頸縮斷裂現象，其中 6 束留有較長心線
  - ◇ 留於錨頭上之殘線嚴重鏽蝕且部分有心線抽出現象







其他：

- 線中央位置有 3 處退絞且鋼線脫離凸出。



### 3. 顯著受損狀況分類及分布整理

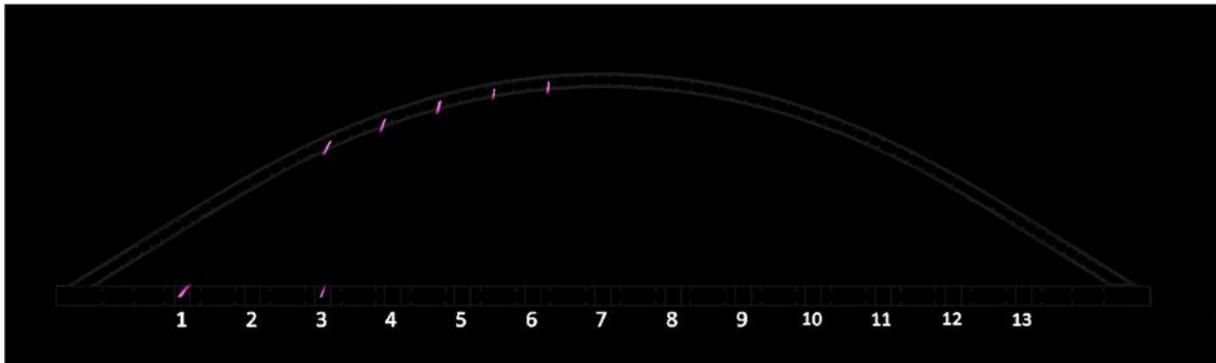


圖 7、鋼絞線末段完整且脫離錨頭

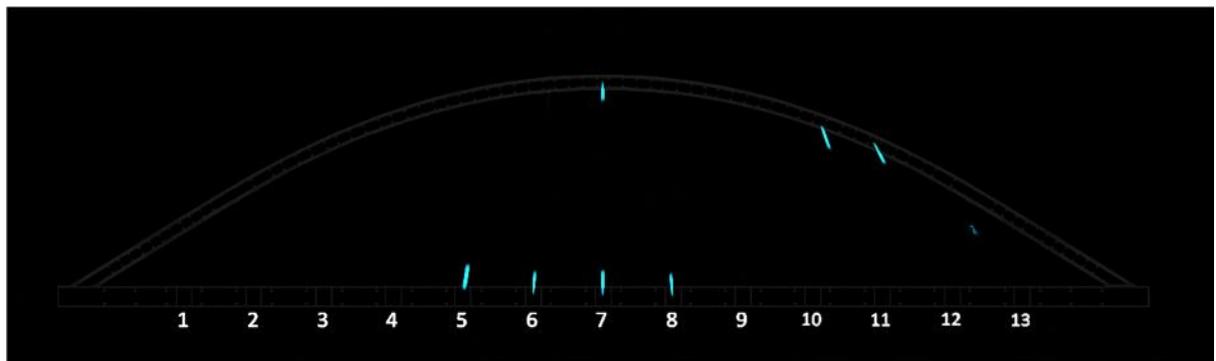


圖 8、鳥籠狀況

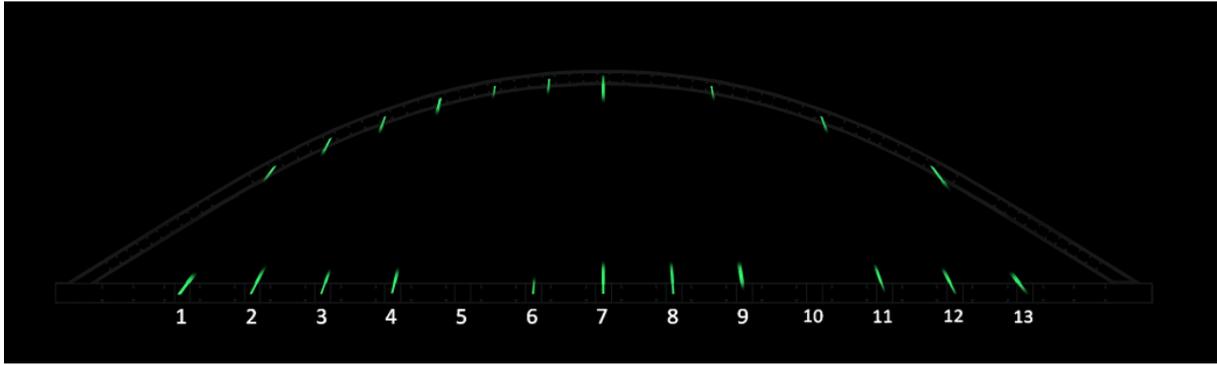


圖 9、退絞狀況

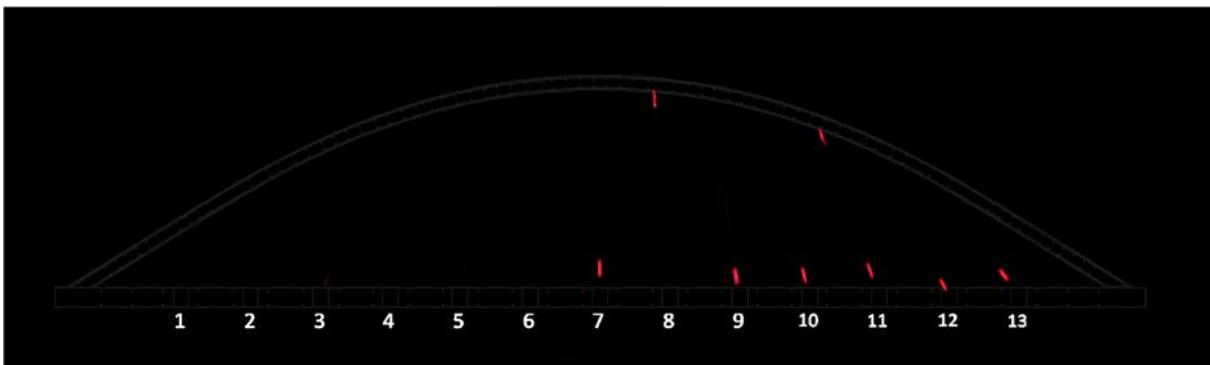


圖 10、頸縮斷裂狀況

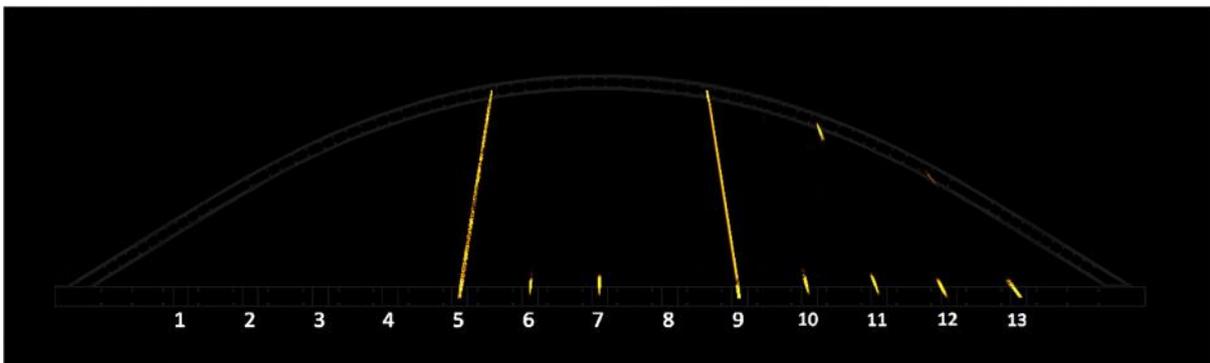


圖 11、鏽蝕狀況

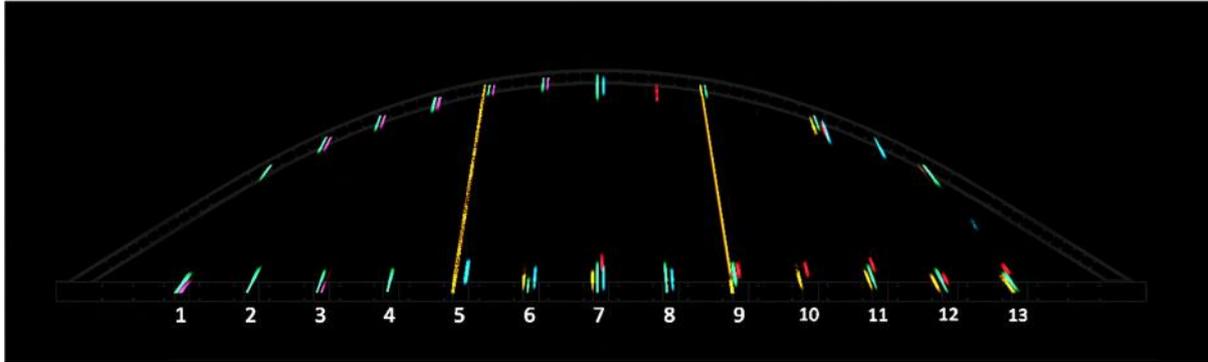


圖 12、吊索顯著破壞狀況總覽

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# 南方澳大橋鋼箱梁 現場勘察報告

委託單位：國家運輸安全調查委員會

執行單位：臺大嚴慶齡工業研究中心

計畫主持人：臺灣大學土木工程系 [REDACTED]

參與人員：臺灣大學土木工程系研究生 [REDACTED]

2020 年 3 月 31 日

## 第 1 章 南方澳大橋鋼箱梁現場勘察簡介

2019 年 10 月 1 日上午 9 時 30 分，位於臺灣宜蘭縣蘇澳鎮南方澳的南方澳大橋發生橋梁斷裂事故，斷裂橋體坍塌造成多名人員傷亡及船隻、車輛損壞。本團隊受國家運輸安全調查委員會委託，由臺灣大學土木工程系 [REDACTED] 教授帶領博士生一名 ([REDACTED])、碩士生兩名 ([REDACTED])，進行現場勘察及資料蒐集。

本團隊負責橋鋼箱梁之勘察，根據記錄橋樑斷裂過程的影片可得知主要斷裂面在橋鋼箱梁中央位置，因此本團隊針對破裂面 (C3-2 節塊與 C4 節塊連接斷面) 進行完整的記錄，同時也另記錄兩個被裁切斷面 (C6 節塊與 C7 節塊連接斷面、C9-1 與 C9-2 節塊連接斷面)，並根據現場勘察鋼箱梁的受損情形規劃檢測計畫，包含拉力試驗與 3 點荷載疲勞試驗之試體可能的位置及實驗方式。

### 1.1 勘察紀錄

本團隊於 2019 年 11 月 29 日、12 月 4 日及 12 月 25 日，分三次前往宜蘭南方澳大橋現場進行拍照、量測與記錄破裂面處破壞細節，以下簡述勘察記錄之過程。

11 月 29 日本團隊首次前往宜蘭進行現地勘察，損壞之南方澳大橋橋身已於事故發生後拆除完成，分成不同節塊置於岸上，節塊斷面如圖 1.1 所示，鋼箱梁以 U 型加勁板來加勁橋板，型式與美國新舊金山奧克蘭海灣大橋相似[1]，這次主要現場勘察重點集中在破裂面以及裁切面的數據蒐集，同時也將斷面進行編號以利進行討論。測量方法是以游標卡尺及厚度計量測 U 型加勁板與鋼箱梁鋼板厚度，因為破裂面已非直線，故拉尼龍繩量測距離並記錄其位置，同時也量測沿切斷面的數據進行比較。

12 月 4 日本團隊進行第二次現地勘察，除了完成斷面的數據蒐集，這一次主要的現場勘察重點在於觀察各節塊 U 型加勁板銲接情況，加勁板銲接情況可分為現地銲接接頭及工廠銲接接頭，由於兩種銲接型式可能造成銲接接頭強度的差異，且現地銲接接頭方式與竣工圖不同，故此此次勘察也在不同鋼箱節塊中尋找

完整的 U 型加勁板及橋板，標記做為未來試體實驗的依據。

12 月 25 日本團隊進行第三次現地勘察，本次勘察重點集中在實驗試體的確認，針對現地銲接接頭、工廠銲接接頭與完整鋼橋母材等三種不同型式的試體種類，選擇適合進行拉力試驗與 3 點荷載疲勞試驗之橋板與 U 型加勁板的試體，確認各試體的外觀形狀是否符合試驗需求，並量測與標定出各試體的長度及寬度，此外也規劃可進行鋼板拉伸試驗之試片(CNS 2112, 2005 [2])，以及將破裂面以及完整面之數據與照片進行完整紀錄。

## 1.2 節塊編號

由於損壞鋼箱橋身已拆除並分成不同節塊置於岸上，如照片 1.1 所示，在現地勘察時現場各不同節塊已有編碼，編號順序與相對位置及方位如圖 1.2 所示，由西側至東側分別為 C1-1、C1-2、C2-1、C2-2、C3-1、C3-2、C4、C5、C6、C7、C8、C9-1、C9-2 等共 13 個編號，本團隊以此編號作為基礎，將每一個節塊的兩個斷面命名為 A 斷面與 B 斷面，A 斷面為節塊東側斷面、B 斷面為節塊西側斷面（如圖 1.2）。將斷面上的 U 型加勁板分為頂部（T）、底部（B）與垂直（V）三種，編號方式由南側開始起算，各 U 型加勁板分別以 T1~T23、B1~B11、V1~V4 來表示（如圖 1.3、圖 1.4），

## 1.3 報告內容

本報告第 1 章簡介南方澳大橋鋼箱梁現場勘察之過程；第 2 章整理破裂斷面（C3-2 節塊與 C4 節塊連接斷面）以及完整斷面（C6 節塊與 C7 節塊連接斷面、C9-1 與 C9-2 節塊連接斷面）其 U 型加勁板及橋板的各項量測數據；第 3 章規劃試驗的檢測計畫，為瞭解各處不同銲接型式的 U 型加勁板強度與母材強度，本團隊規劃多組不同銲接形式、不同位置的試體，選取足夠數量的構件進行拉力試驗以及 3 點荷載疲勞試驗；第 4 章結論與建議。



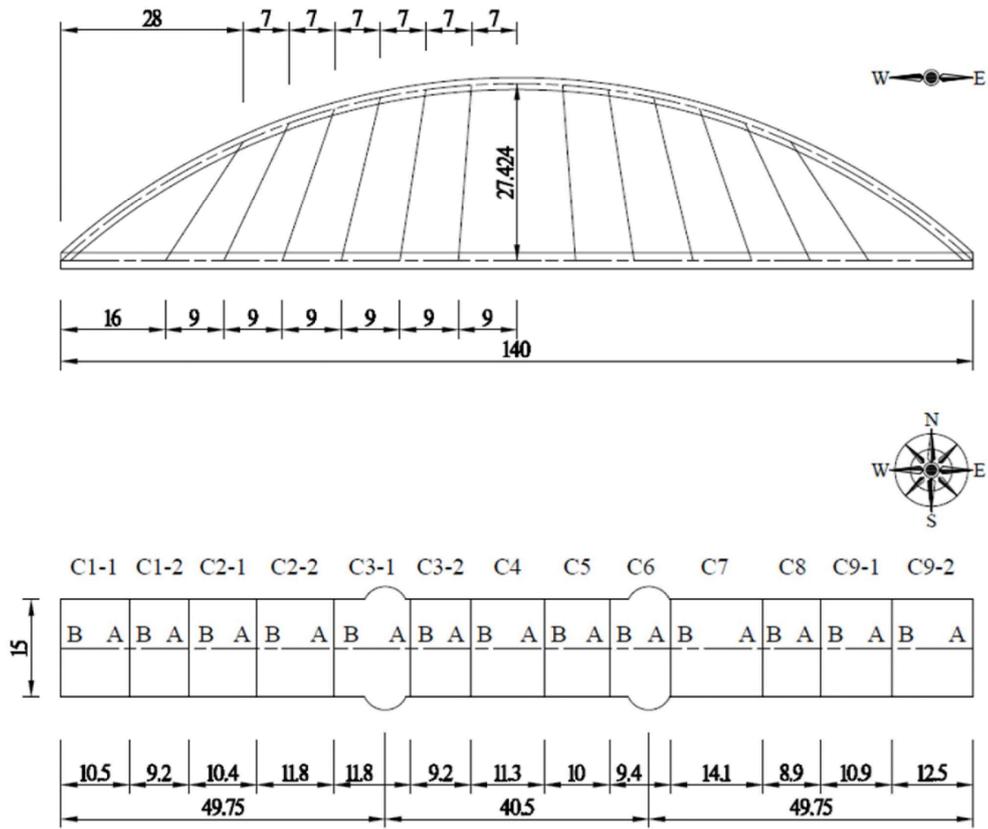


圖 1.2 橋梁立面圖、平面圖與節塊編號

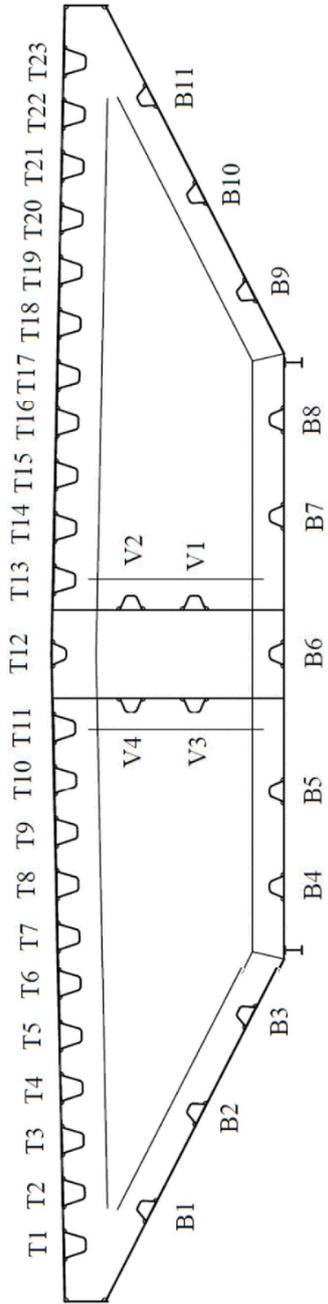


圖 1.3 断面 A

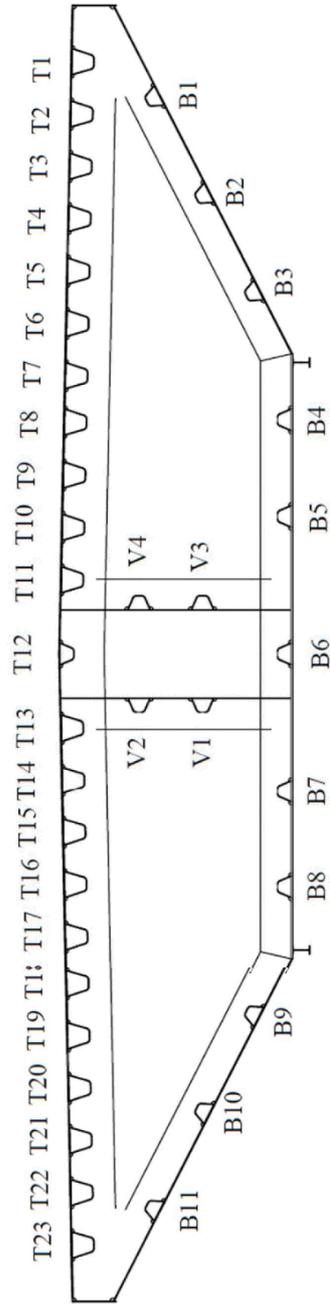


圖 1.4 断面 B



(a) C1-1 節塊南側



(b) C1-2 節塊南側



(c) C2-1 節塊南側



(d) C2-2 節塊南側



(e) C3-1 節塊南側



(f) C3-2 節塊南側



(g) C4 節塊南側



(h) C5 節塊南側

照片 1.1 現場各節塊與其編號



(i) C6 節塊南側



(j) C7 節塊南側



(k) C8 節塊南側



(l) C9-1 節塊南側



(m) C9-2 節塊南側

照片 1.1 現場各節塊與其編號(續 1)

## 第 2 章 南方澳大橋鋼箱梁現場勘察結果

南方澳大橋鋼主梁斷面如圖 1.1 所示，鋼箱梁上橋板與下橋板厚度皆為 12 mm，側邊底部鋼板厚度為 10 mm，中央兩片腹板厚度為 10 mm，上橋板下方共有 23 組 U 型加勁板，下橋板及側邊鋼板上方則為 11 組 U 型加勁板，腹板處有 4 組 U 型加勁板。現場勘察共取三個斷面進行量測，分別為(1) C3-2 節塊連接 C4 節塊之斷面，本斷面為南方澳大橋鋼箱梁破壞之斷面，詳細紀錄斷面上鋼板的厚度變化及破壞型式，(2) C6 節塊連接 C7 節塊之斷面與(3) C9-1 節塊連接 C9-2 節塊之斷面，此兩斷面為取得軸力試驗試體與三點彎曲試驗試體之主要斷面，因此亦詳加紀錄。

現場勘察情形如照片 2.1 所示，量測過程如照片 2.2 所示，三個斷面都進行上橋板、下橋板、側邊鋼板、腹板與 U 型加勁板的厚度量測，鋼箱梁斷面上橋板、下橋板與側邊鋼板的厚度變化量測方式為，於 U 型加勁板下方及 U 型加勁板間各取一點，量測鋼板的厚度並記錄其位置。U 型加勁板之厚度量測方式為，於 U 型加勁板兩側及 U 型底部可進行量測之位置取點進行量測，其編號位置如圖 2.1 所示，此外針對 C3-2 節塊連接 C4 節塊之斷面，因其為鋼箱梁破壞之斷面，下橋板的鋼板破壞型式、破壞形狀與長度也進行量測並紀錄。

### 2.1 破裂斷面量測紀錄

C3-2 節塊與 C4 節塊連接處之斷面為南方澳大橋鋼箱梁破壞之斷面，此斷面共有兩側，C3-2 節塊接 C4 節塊側斷面(C3-2A 斷面，照片 2.3)及 C4 節塊接 C3-2 節塊側斷面(C4B 斷面，照片 2.4)，此斷面位於鋼箱梁施工時節塊續接之位置。加勁板編號方式如圖 1.3 與圖 1.4 所示，為 A 斷面上方 U 型加勁板編號為 AT1~AT23，下方 U 型加勁板編號為 AB1~AB11，B 斷面上方 U 型加勁板編號為 BT1~BT23，下方 U 型加勁板編號為 BB1~BB11，垂直向 U 型加勁板編號為 AV1~AV4 及 BV1~BV4。U 型加勁板間鋼箱梁下橋板厚度之量測位置如圖 2.2 與圖 2.3 所示，厚度量測結果列於表 2.1 與表 2.2，量測位置可參考圖 2.2 及圖 2.3

所示，以下概述各 U 型加勁板與梁翼板之厚度紀錄過程。

### 2.1.1 鋼箱梁斷面下方之 U 型加勁板(AB1~AB11、BB1~BB11)與橋板厚度

以下敘述各加勁板與橋板觀察記錄情形：

#### ■ AB1 與 BB1

AB1 與 BB1 斷面如照片 2.5(a)與照片 2.5(b)所示，A 側與 B 側下方鋼板厚度分別為 8.9 mm 及 9.4 mm，U 型加勁板厚度為 8.9 mm。

#### ■ AB2 與 BB2

AB2 與 BB2 斷面如照片 2.5(c)與照片 2.5(d)所示，A 側與 B 側下方鋼板厚度分別為 9.0 mm 及 10.3 mm，U 型加勁板厚度 9.7 mm(包含銲道)。銲道無背墊板。

#### ■ AB3 與 BB3

AB3 與 BB3 斷面如照片 2.5(e)與照片 2.5(f)所示，A 側因有端板下方鋼板厚度無法量測，端板旁鋼板厚度為 9.4 mm，B 側下方鋼板厚度 9.9 mm，U 型加勁板厚度 8.8 mm。A 側 U 型加勁板與端板間有明顯縫隙，且銲道無背墊板。

#### ■ AB4 與 BB4

AB4 與 BB4 斷面如照片 2.5(g)與照片 2.5(h)所示，A 側下方鋼板厚度為 10.7 mm，U 型加勁板厚度為 9.1 mm(包含銲道)。B 側緊貼地面因此無法量測。

#### ■ AB5 與 BB5

AB5 與 BB5 斷面如照片 2.5(i)與照片 2.5(j)所示，A 側下方鋼板厚度為 10.8 mm，U 型加勁板厚度為 9.2 mm(包含銲道)。B 側 U 型加勁板旁鋼板厚度為 11.4 mm。

#### ■ AB6 與 BB6

AB6 斷面如照片 2.5(k)所示，此 U 型加勁板斷裂後僅在 A 側有加勁板，B 側無 U 型加勁板，量測加勁板底部無銲道處鋼板厚度為 8.4 mm。照片 2.5(l)顯示 AB6 加勁板之內部銲接，於 U 型加勁板與端板間有明顯縫隙。

#### ■ AB7 與 BB7

AB7 與 BB7 斷面如照片 2.5(m)與照片 2.5(n)所示，A 側下方鋼板厚度為 12

mm，U 型加勁板厚度為 8.5 mm，B 側有端板無法量測 U 型加勁板厚度。

■ AB8 與 BB8

AB8 與 BB8 斷面如照片 2.5(o)與照片 2.5(p)所示，A 側有端板無法量測，B 側下方鋼板厚度為 11.4 mm，U 型加勁板厚度為 9.3 mm。

■ AB9 與 BB9

AB9 與 BB9 斷面如照片 2.5(q)與照片 2.5(r)所示，A 側有端板無法量測，B 側下方鋼板厚度為 9.6 mm，U 型加勁板厚度為 9.9 mm。

■ AB10 與 BB10

AB10 與 BB10 斷面如照片 2.5(s)與照片 2.5(t)所示，A 側下方鋼板厚度為 9.1 mm，U 型加勁板厚度為 8.5 mm，U 型加勁板與下方鋼板間有明顯空隙(照片 2.5(u)與照片 2.5(v))，左側空隙為 6.6 mm，右側空隙為 7.35 mm。B 側有端板無法量測。

■ AB11 與 BB11

AB11 與 BB11 斷面如照片 2.5(w)與照片 2.5(x)所示，A 側下方鋼板厚度為 9.2 mm，U 型加勁板厚度為 8.4 mm，U 型加勁板與下方鋼板間有明顯空隙(照片 2.5(y)與照片 2.5(z))，左側空隙為 4.7 mm，右側空隙為 6.5 mm。

**2.1.2 鋼箱梁斷面腹板之 U 型加勁板(AV1~AV4、BV1~BV4)與腹板厚度**

編號 AV1~AV4 與 BV1~BV4 之 U 型加勁板如照片 2.6(a)與照片 2.6(b)所示，位於鋼箱梁腹板上。

■ AV1、BV1、AV2、BV2

AV1、BV1、AV2、BV2 之側面圖如照片 2.6(c)與照片 2.6(d)所示，這兩組 U 型加勁板都自鐸道續接處斷裂，AV1 與 BV1 下方鋼箱梁腹板厚度分別為 9.4 mm 及 9 mm，AV1 之 U 型加勁板厚度為 8.8 mm，AV2 與 BV2 之 U 型加勁板厚度分別為 9.2 mm 與 9.4 mm。量測 U 型加勁板斷裂處 U 型加勁板及分離之鋼梁腹板長度，BV1 處鋼梁腹板長度為 36 cm(照片 2.6(g))，AV1 之 U 型加勁板長度為 33 cm(照片 2.6(h))，推算此處鋼梁腹板伸長率為 9%，BV2 處鋼梁腹板長度為 27

cm(照片 2.6(i))，AV1 之 U 型加勁板長度為 23 cm(照片 2.6(j))，推算此處鋼梁腹板伸長率為 17%。AV1 與 BV1 處鋼梁腹板呈現斜斷面之破壞(照片 2.6(k)與照片 2.6(l))。

■ AV3、BV3、AV4、BV4

AV3、BV3、AV4、BV4 之側面圖如照片 2.6(e)與照片 2.6(f)所示，BV3 下方鋼箱梁腹板厚度為 9.5 mm，AV3 與 AV4 之 U 型加勁板厚度無法量測，BV3 與 BV4 之 U 型加勁板厚度分別為 8.9 mm 與 9.0 mm。

**2.1.3 鋼箱梁斷面上方之 U 型加勁板(AT1~AT23、BT1~BT23)與橋板厚度**

斷面上方 U 型加勁板與橋板破壞嚴重，許多 U 型加勁板無法進行量測，而橋板皆因變形無法量測，記錄到的資訊如下所列：

■ AT1~AT3 與 BT1~BT3

AT1~AT3 與 BT1~BT3 如照片 2.7(a)與照片 2.7(b)所示，B 側編號 BT2 之 U 型加勁板厚度為 8.8 mm，A 側編號 AT3 之 U 型加勁板厚度為 9.1 mm，其餘因變形無法量測。

■ AT4~AT6 與 BT4~BT6

AT4~AT6 與 BT4~BT6 如照片 2.7(c)與照片 2.7(d)所示，A 側編號 AT4 之 U 型加勁板厚度為 8.7 mm，其餘因變形無法量測。

■ AT7~AT9 與 BT7~BT9

AT7~AT9 與 BT7~BT9 如照片 2.7(e)與照片 2.7(f)所示，兩側的 U 型加勁板皆因變形而無法量測。

■ AT10~AT11 與 BT10~BT11

AT10~AT11 與 BT10~BT11 如照片 2.7(g)與照片 2.7(h)所示，B 側編號 BT10 之 U 型加勁板厚度為 8.7 mm，A 側編號 AT11 之 U 型加勁板厚度為 9.1 mm。

■ AT12 與 BT12

AT12 與 BT12 如照片 2.6(a)與照片 2.6(b)所示，此 U 型加勁板之尺寸比其他上端 U 型加勁板小，A 側因變形無法量測，B 側有端板而無法量測。

■ AT13~AT15 與 BT13~BT15

AT13~AT15 與 BT13~BT15 如照片 2.7(i)與照片 2.7(j)所示，A 側編號 AT13 之 U 型加勁板厚度為 9.6 mm，B 側編號 BT14 之 U 型加勁板厚度為 8.9 mm，編號 BT15 之 U 型加勁板厚度為 9.0 mm。編號 T14 之 U 型加勁板同時於 A、B 兩側斷裂掉落於地面(照片 2.7(k))。

■ AT16~AT18 與 BT16~BT18

AT16~AT18 與 BT16~BT18 如照片 2.7(m)與照片 2.7(n)所示，B 側 U 型加勁板 BT18 之厚度為 8.8 mm，其餘因變形無法量測。編號 T18 之 U 型加勁板同時於 A、B 兩側斷裂掉落於地面(照片 2.7(l))。

■ AT19~AT21 與 BT19~BT21

AT19~AT21 與 BT19~BT21 如照片 2.7(o)與照片 2.7(p)所示，B 側編號 BT20 之 U 型加勁板厚度為 8.9 mm，編號 BT21 之 U 型加勁板厚度為 8.8 mm。

■ AT22~AT23 與 BT22~BT23

AT22~AT23 與 BT22~BT23 如照片 2.7(q)與照片 2.7(r)所示，兩側的 U 型加勁板皆因變形而無法量測。

## 2.2 破裂斷面破壞型式

破裂型式之紀錄以 C4 節塊接 C3-2 南節塊側斷面(C4B 斷面)為主，量測結果如圖 2.4 所示，其斷面經過之鐸道已標於圖上，並分為斜斷面(藍色虛線)以及頸縮斷面(紅色實線)，此外亦對 C3-2 節塊側(C3-2A 斷面)進行拍照記錄。以下概述斷面破裂型式之觀察紀錄。

### C4B 斷面

■ 外側至 B1 間

從 C4 節塊斷裂面外側至 B1 間幾乎皆為斜斷面，除 B1 下側有些許頸縮斷面，如照片 2.8(a)所示。

■ B1 至 B2 間

從 B1 至 B2 之間如照片 2.8(b)所示，斷裂面轉折向內，其斷面為頸縮斷面。

■ B2 至 B3 間

B2 至 B3 間大部份為斜斷面，僅 B2 下側與 B2、B3 中段(如照片 2.8(c)與 2.8(d)所示)有些許頸縮斷面。

■ B3 至 I 型梁間

B3 下側為斜斷面，至 I 型梁前斷裂面轉折向外，其斷面為頸縮斷面(如照片 2.8(e)所示)。

■ I 型梁至 B5 間

從 I 型梁至 B4 前其斷面為頸縮斷面，而 B4 下側至 B5 下側均為斜斷面。

■ B5 至 B7 間

B5 與 B7 往中間 B6 方向，斷裂面均轉折向外延伸至垂直版，其轉折處斷面為頸縮斷面，其餘皆為斜斷面。

■ B7 至 I 型梁間

B7 至 I 型梁間如照片 2.8(f)所示，僅有一小段呈現頸縮斷面，而 B8 至 I 型梁間斷裂面轉折向內，其斷面為頸縮斷面，其餘皆為斜斷面。

■ I 型梁至 B9 間

如照片 2.8(g)所示，從 I 型梁至 B9 前斷裂面轉折向外，其斷面為頸縮斷面，其餘皆為斜斷面。

■ B9 至 B11 間

B10 至外側間如照片 2.8(h)所示，僅有一小段呈現頸縮斷面，其餘皆為斜斷面。而 B11 至 C4 節塊斷裂面外側，因斷面撕裂嚴重故無法辨識。

**C3-2A 斷面**

如照片 2.9(a)所示，AB3 與 AB4 間有鋼板頸縮斷面，I 型梁上方為鋼板的續接位置(照片 2.9(b))，如照片 2.9(c)所示，I 型梁上方鋼板呈現垂直斷裂面，與其他位置所呈現的斷裂型式不同，AB4 與 AB5 間有鋼板頸縮斷面與斜斷面(照片 2.9(d)與 2.9(e))，與另一側相同，AB8 與 AB9 間 I 型梁上方為鋼板的續接位置(照

片 2.9(f))。

## 2.3 完整斷面量測紀錄

### C6 節塊與 C7 節塊連接斷面

在第三章的試體規劃中，許多試體皆取自 C6 節塊與 C7 節塊，因此於此連接斷面兩側(C6A 斷面與 C7B 斷面)進行量測(照片 2.10(a))，上橋板、下橋板、側邊鋼板、腹板之量測項目如圖 2.5 所示；上方 U 型加勁板編號 T16 之後皆因焰切變形嚴重無法進行量測(照片 2.10(b))，因此選取 T1、T2、T4、T5、T8 及 T15 進行量測(照片 2.10(c)至照片 2.10(j))，如照片 2.10(k)至照片 2.10(n)所示，下方 U 型加勁板選取 B1、B4、B7 及 B9 進行量測，垂直 U 型加勁板選取 V1 及 V4 進行量測，U 型加勁板與橋板之板厚度量測結果列於表 2.3 與表 2.4。

### C9-1 節塊與 C9-2 節塊連接斷面

第三章中許多試體取自 C9-1 節塊與 C9-2 節塊，因此於此連接斷面兩側(C9-1A 斷面與 C9-2B 斷面)進行量測(照片 2.11(a)與照片 2.11(b))，上橋板、下橋板、側邊鋼板、腹板之量測項目如圖 2.6 所示；如照片 2.11(c) 至照片 2.11(j)所示，上方 U 型加勁板選取 T4、T8、T17 及 T20 進行量測，下方 U 型加勁板選取 B1、B4、B8 及 B10 進行量測(照片 2.11(k)至照片 2.11(p))，垂直 U 型加勁板選取 V1 及 V3 進行量測(照片 2.11(q)至照片 2.11(t))，U 型加勁板與橋板之板厚度量測結果列於表 2.5 與表 2.6。

表 2.1 C3-2A 斷面鋼箱梁厚度量測結果

		C3-2A 斷面 U 型加勁板												
Rib	T3	T4	T11	T13	B1	B2	B4	B5	B6	B7	B10	B11	V1	V2
厚度(mm)	9.1	8.7	9.1	9.6	8.9	9.7	9.1	9.2	8.4	8.5	8.5	8.4	8.8	9.2

		C3-2A 斷面橋板										
點位	b1	b2	b3	b4	b5	b6	b7	b8	b9	b10	b11	
板厚(mm)	9.2	8.9	8.3	9.0	8.1	9.4	9.1	11.9	10.7	10.5	10.8	
點位	b12	b13	b14	b15	b16	b17	b18	b19	b20	b21	v1	
板厚(mm)	10.5	12.0	10.7	10.8	9.4	8.8	9.1	8.8	9.2	8.8	9.4	

表 2.2 C4B 斷面鋼箱梁厚度量測結果

		C4B 斷面 U 型加勁板												
Rib	T2	T10	T14	T15	T18	T20	T21	B3	B8	B9	V2	V3	V4	
厚度(mm)	8.8	8.7	8.9	9.0	8.8	8.9	8.8	8.8	9.3	9.9	9.4	8.9	9.0	

		C4B 斷面橋板										
點位	t1	t2	b1	b2	b3	b4	b5	b6	b7	b8	b9	
板厚(mm)	12.2	12.3	9.4	9.2	10.3	9.3	9.9	8.5	11.6	11.4	10.4	
點位	b10	b11	b12	b13	b14	b15	b16	b17	b18	v1	v2	
板厚(mm)	11.4	11.3	8.5	9.6	8.5	8.5	9.0	9.0	8.9	9.0	9.5	

表 2.3 C6A 斷面鋼箱梁厚度量測結果

Rib	頂部與底部 U 型加勁板										垂直 U 型加勁板		
	T1	T2	T4	T5	T8	T15	B1	B4	B7	B9	Rib	V1	V4
頂部(mm)	8.9	9.0	9.0	9.0	8.6	9.0	8.6	8.8	9.1	9.2	頂部(mm)	9.0	8.6
S 側(mm)	8.6	8.6	8.9	8.6	8.7	8.4	8.7	8.6	9.8	8.8	上側(mm)	8.8	8.8
N 側(mm)	8.9	8.9	8.6	8.8	8.6	8.6	8.8	8.7	9.4	8.9	下側(mm)	8.7	8.7

點位	上橋板、下橋板與腹板															
	t1	t2	t3	t4	t5	t6	b1	b2	b3	b4	b5	b6	b7	b8	v1	v2
板厚(mm)	12.3	12.4	12.7	12.4	12.5	13.0	10.7	11.0	12.1	12.7	13.0	13.0	11.0	10.3	10.5	10.5

表 2.4 C7B 斷面鋼箱梁厚度量測結果

Rib	頂部與底部 U 型加勁板										垂直 U 型加勁板		
	T1	T2	T4	T5	T8	T15	B1	B4	B7	B9	Rib	V1	V4
頂部(mm)	8.9	8.8	9.1	9.1	8.8	8.9	8.7	8.7	9.1	9.0	頂部(mm)	9.0	8.5
N 側(mm)	8.7	8.7	8.7	8.6	8.6	8.9	8.7	8.7	9.2	9.4	上側(mm)	8.7	8.7
S 側(mm)	8.8	8.6	9.0	8.8	8.7	8.7	9.0	8.8	9.1	9.1	下側(mm)	8.7	8.7

點位	上橋板、下橋板與腹板															
	t1	t2	t3	t4	t5	t6	b1	b2	b3	b4	b5	b6	b7	b8	v1	v2
板厚(mm)	12.3	12.5	12.7	12.6	12.8	12.5	10.7	11.5	12.9	12.7	12.9	13.1	10.9	9.8	10.5	10.5

表 2.5 C9-1A 斷面鋼箱梁厚度量測結果

Rib	頂部與底部 U 型加勁板								垂直 U 型加勁板		
	T4	T8	T17	T20	B1	B4	B8	B10	Rib	V1	V3
頂部(mm)	9.7	8.7	8.4	8.8	9.3	8.5	9.3	8.7	頂部(mm)	8.7	8.4
S 側(mm)	8.5	8.6	9.1	8.7	9.9	9.2	8.3	8.5	上側(mm)	8.8	8.6
N 側(mm)	8.9	8.6	9.2	8.2	9.2	8.5	8.3	8.6	下側(mm)	8.3	8.5

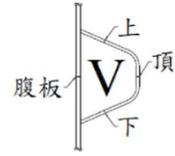
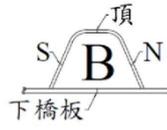
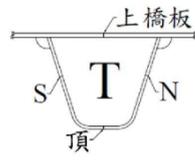
上橋板、下橋板與腹板										
點位	t1	t2	t3	t4	b1	b2	b3	b4	v1	v2
板厚(mm)	13.6	13.0	12.6	12.9	11.6	12.5	12.7	11.3	10.6	11.0

表 2.6 C9-2B 斷面鋼箱梁厚度量測結果

Rib	頂部與底部 U 型加勁板								垂直 U 型加勁板		
	T4	T8	T17	T20	B1	B4	B8	B10	Rib	V1	V3
頂部(mm)	8.5	8.4	8.7	8.8	9.2	8.5	8.7	8.4	頂部(mm)	8.4	8.6
S 側(mm)	8.3	9.0	8.5	8.6	10.1	8.7	8.3	8.6	上側(mm)	8.6	8.7
N 側(mm)	8.7	8.5	8.7	8.9	9.0	8.7	8.5	8.8	下側(mm)	8.4	8.5

上橋板、下橋板與腹板										
點位	t1	t2	t3	t4	b1	b2	b3	b4	v1	v2
板厚(mm)	13.3	13.6	12.6	12.9	11.0	12.9	12.7	11.2	10.9	10.5

A斷面



B斷面

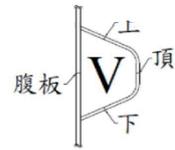
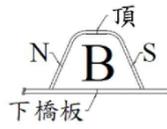
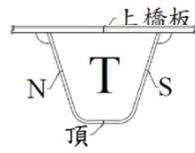


圖 2.1 U 型加勁板厚度量測位置

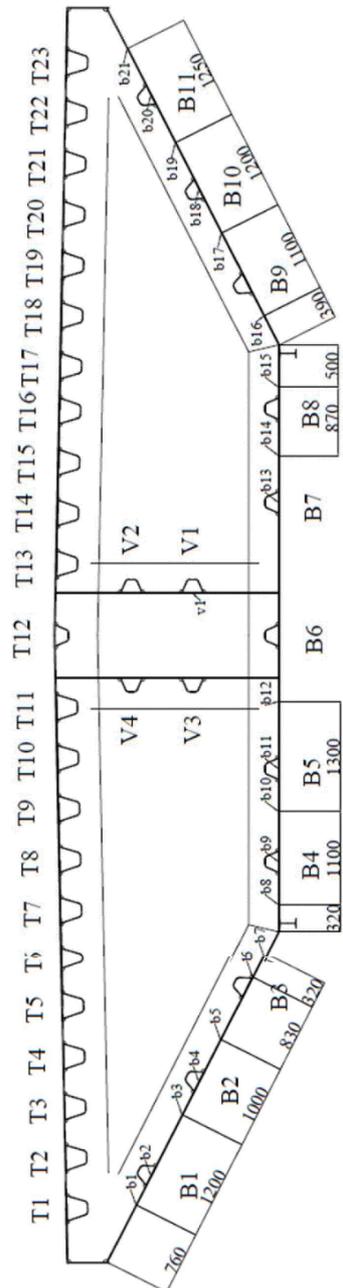


圖 2.2 C3-2A 斷面 U 型加勁板間梁橋板厚度量測位置(單位: mm)

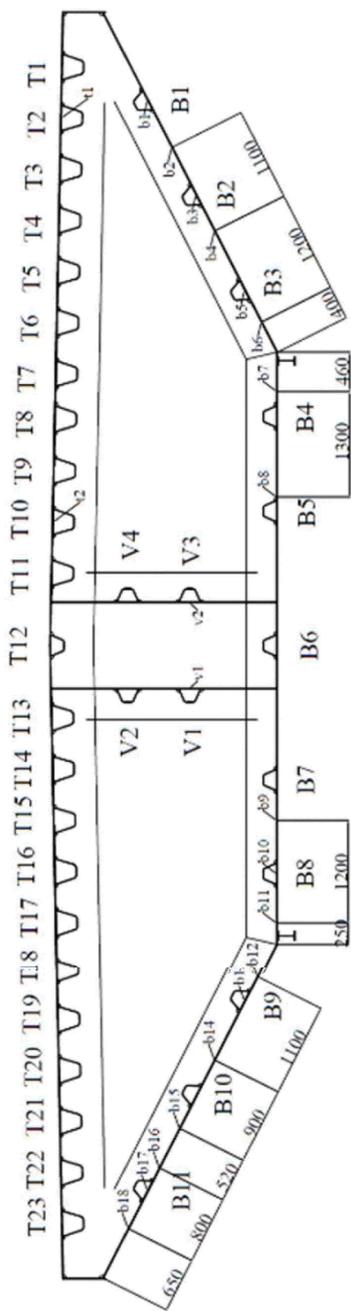


圖 2.3 C4B 斷面 U 型加勁板間梁橋板厚度量測位置(單位: mm)

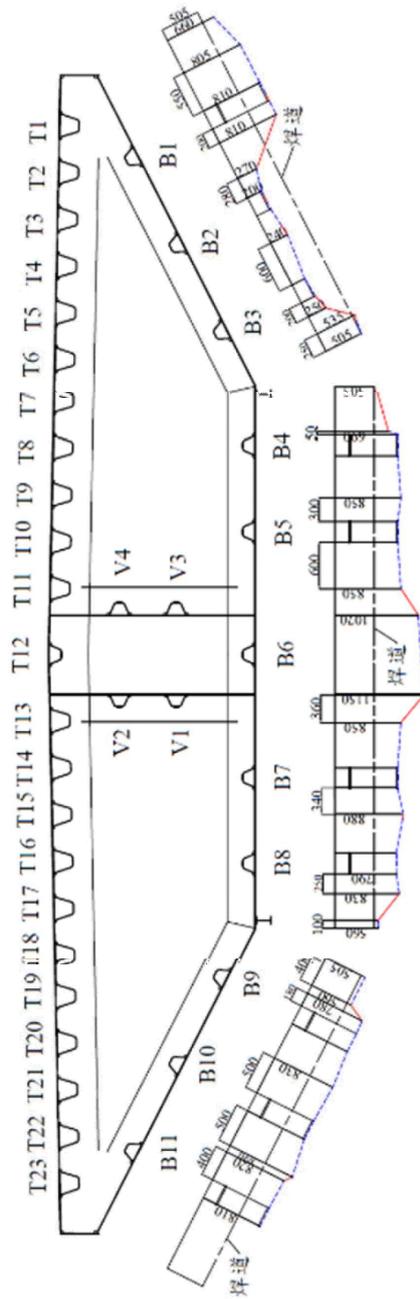


圖 2.4 C4B 斷面底部俯視圖(單位: mm)

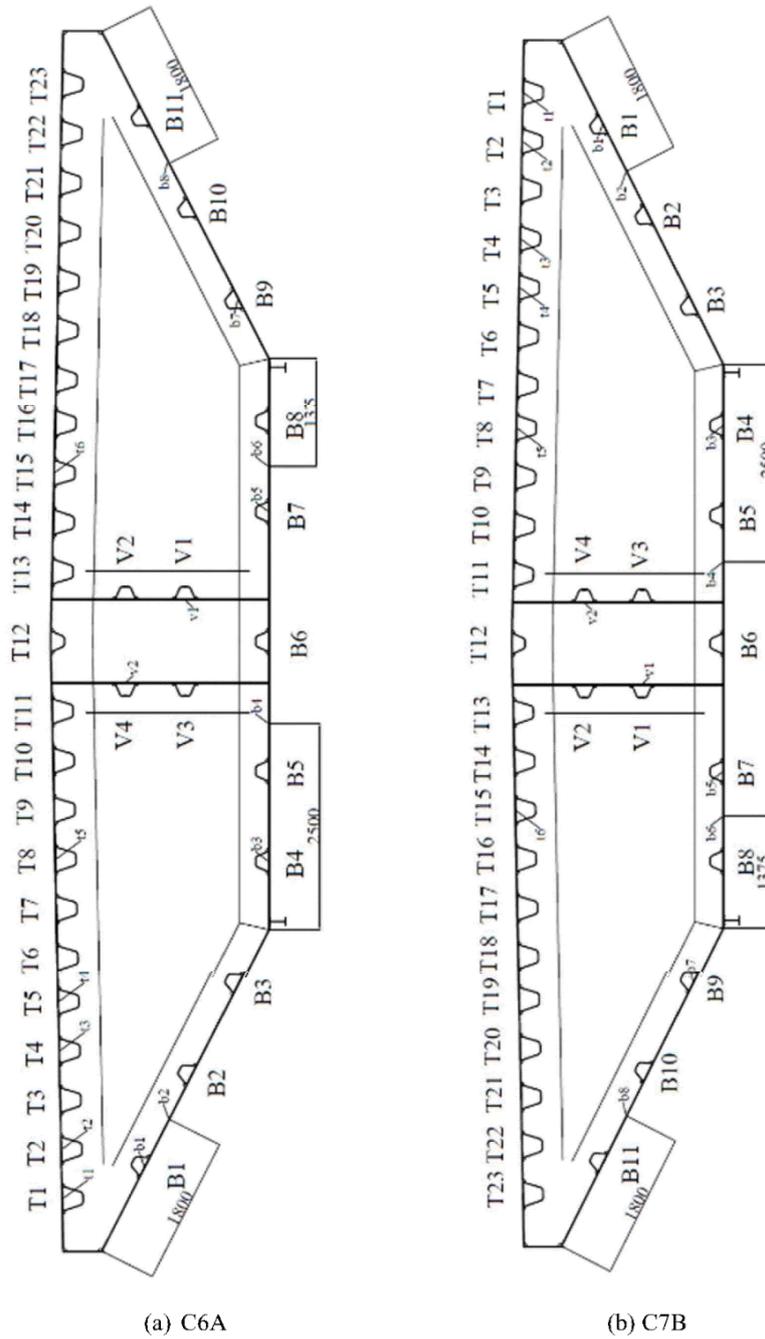


圖 2.5 C6A 與 C7B 斷面 U 型加勁板間梁橋板厚度量測位置(單位: mm)

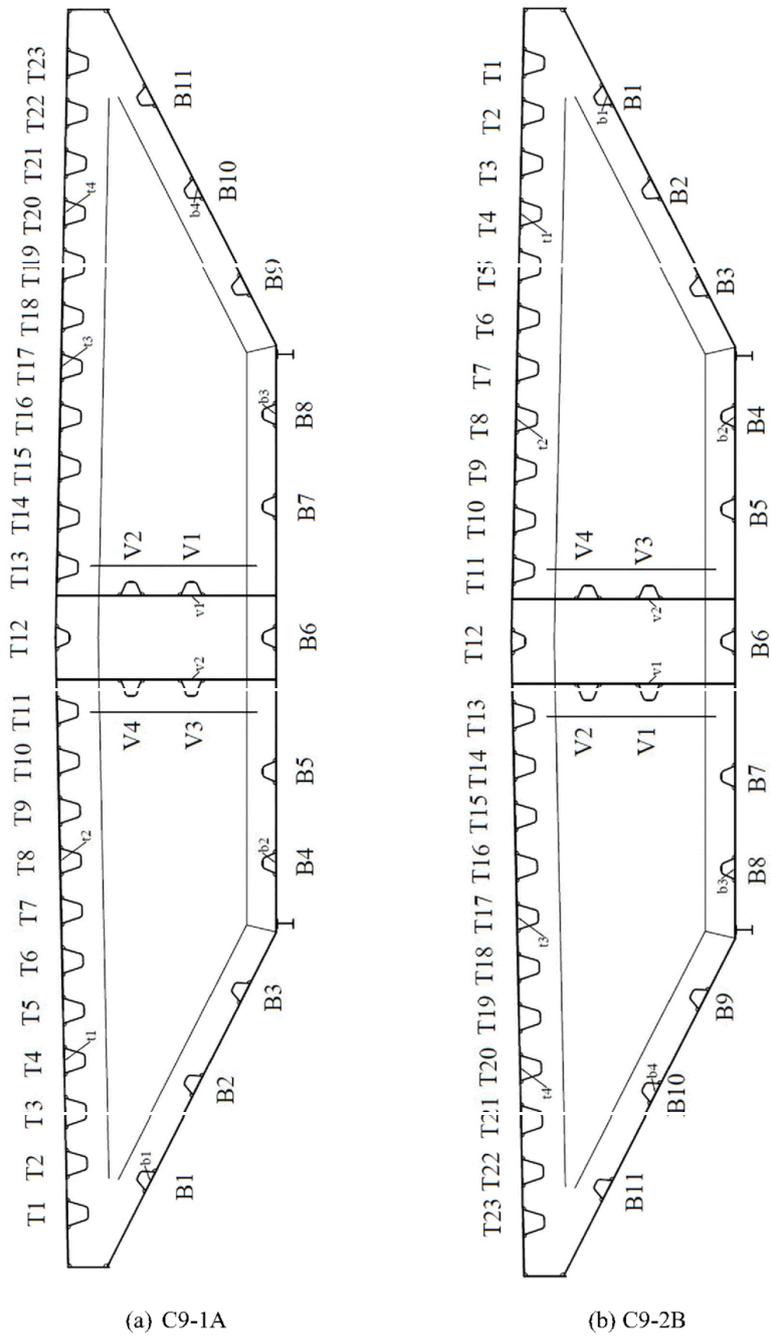


圖 2.6 C9-1A 與 C9-2B 斷面 U 型加勁板間梁橋板厚度量測位置(單位: mm)



照片 2.1 現場勘察



(a) 量測位置



(b) 量測鋼板厚度

照片 2.2 量測方式



照片 2.3 C3-2A 斷面



照片 2.4 C4B 斷面



(a) AB1



(b) BB1



(c) AB2



(d) BB2



(e) AB3



(f) BB3



(g) AB4



(h) BB4

照片 2.5 C3-2 節塊與 C4 節塊相接斷面下方 U 型加勁板



(i) AB5



(j) BB5



(k) AB6



(l) AB6 內側銲接



(m) AB7



(n) BB7



(o) AB8



(p) BB8

照片 2.5 C3-2 節塊與 C4 節塊相接斷面下方 U 型加勁板(續 1)



(q) AB9



(r) BB9



(s) AB10



(t) BB10



(u) AB10 左側與底板空隙



(v) AB10 右側與底板空隙

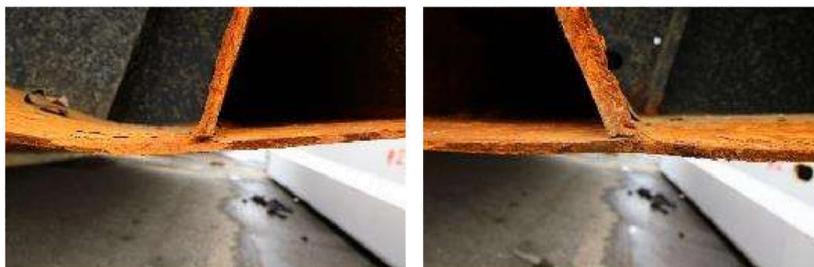


(w) AB11



(x) BB11

照片 2.5 C3-2 節塊與 C4 節塊相接斷面下方 U 型加勁板(續 2)



(y) AB11 左側與底板空隙

(z) AB11 右側與底板空隙

照片 2.5 C3-2 節塊與 C4 節塊相接斷面下方 U 型加勁板(續 3)



(a) AV1~AV4、AT12



(b) BV1~BV4、BT12



(c) BV1, BV2



(d) AV1、AV2



(e) AV3、AV4



(f) BV3、BV4

照片 2.6 C3-2 節塊與 C4 節塊相接斷面垂直 U 型加勁板



(g)BV1



(h)AV1



(i)BV2



(j)AV2



(k)AV1 處腹板斜向破壞



(l)BV1 處腹板斜向破壞

照片 2.6 C3-2 節塊與 C4 節塊相接斷面垂直 U 型加勁板(續)



(a) AT1~AT3



(b) BT1~BT3



(c) AT4~AT6



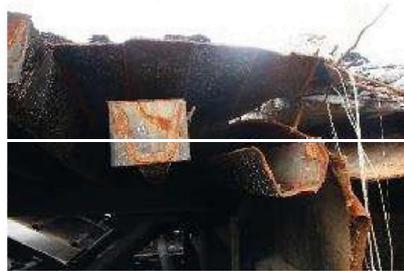
(d) BT4~BT6



(e) AT7~AT9



(f) BT7~BT9



(g) AT10~AT11



(h) BT10~BT11

照片 2.7 C3-2 節塊與 C4 節塊相接斷面上方 U 型加勁板



(i) AT13~AT15



(j) BT13~BT15



(k) BT14 掉落地面



(l) BT18 掉落地面



(m) AT16~AT18



(n) BT16~BT18

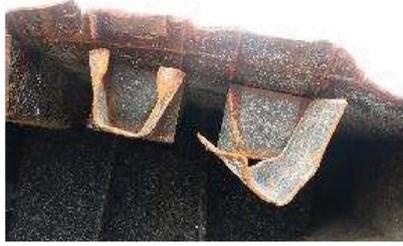


(o) AT19~AT21



(p) BT19~BT21

照片 2.7 C3-2 節塊與 C4 節塊相接斷面上方 U 型加勁板(續 1)



(q) AT22~AT23



(r) BT22~BT23

照片 2.7 C3-2 節塊與 C4 節塊相接斷面上方 U 型加勁板(續 2)



(a) B1 些許頸縮斷面



(b) B1 與 B2 間頸縮斷面



(c) B2 下方頸縮斷面



(d) B2 與 B3 間斜斷面



(e) B3 至 I 型梁



(f) B8 下方鋼版



(g) B9 下方鋼版



(h) B10 至外側

照片 2.8 C4B 斷面下翼板斷裂照



(a) AB3 與 AB4 間鋼板頸縮



(b) 照片 AB3 與 AB4 間鋼板續接



(c) AB3 與 AB4 間垂直向破裂面



(d) AB4 與 AB5 間鋼板頸縮



(e) AB4 與 AB5 間斜破裂面



(f) AB8 與 AB9 間鋼板續接

照片 2.9 C3-2A 斷面下翼板照



(a) C6 節塊與 C7 節塊連接斷面



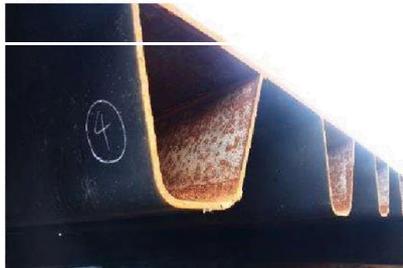
(b) T16 之後鋼板變形



(c) AT1 與 AT2



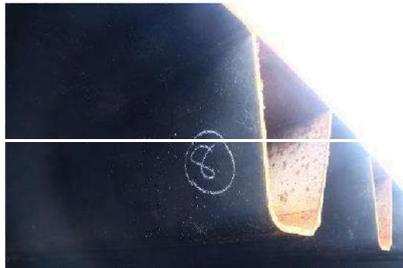
(d) BT1 與 BT2



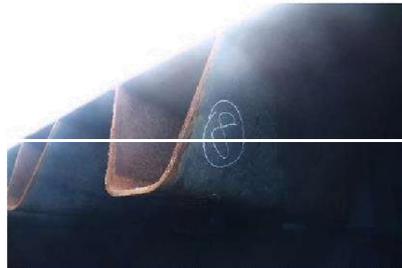
(e) AT4 與 AT5



(f) BT4 與 BT5



(g) AT8



(h) BT8

照片 2.10 C6 節塊與 C7 節塊連接斷面 U 型加勁板



(i) AT15

(j) BT15



(k) AB1 與 BB1



(l) AB4 與 BB4



(m) AB7 與 BB7



(n) AB9 與 BB9

照片 2.10 C6 節塊與 C7 節塊連接斷面 U 型加勁板(續 1)



(a) C9-1 節塊與 C9-2 節塊連接斷面



(b) C9-1 節塊與 C9-2 節塊連接斷面



(c) AT4



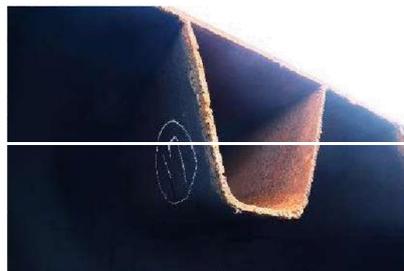
(d) BT4



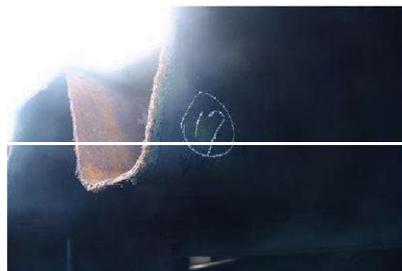
(e) AT8



(f) BT8

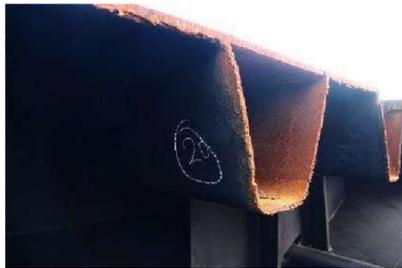


(g) AT17

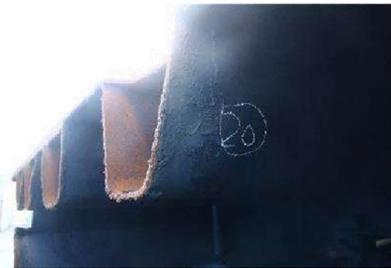


(h) BT17

照片 2.11 C9-1 節塊與 C9-2 節塊連接斷面 U 型加勁板



(i) AT20



(j) BT20



(k) BB1



(l) AB1



(m) BB4



(n) AB4



(o) AB8 與 BB8



(p) AB10 與 BB10

照片 2.11 C9-1 節塊與 C9-2 節塊連接斷面 U 型加勁板(續 1)



(q) BV1



(r) AV1



(s) AV3



(t) BV3

照片 2.11 C9-1 節塊與 C9-2 節塊連接斷面 U 型加勁板(續 2)

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### 附錄 3 鋼絞線試驗報告

TS-11-04

NOVA



華光工程顧問股份有限公司  
CECI Nova Technology Co., Ltd.



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#### 鋼絞線試驗報告

報告編號：20-00103Y

收件日期：109.01.21

報告日期：109.01.30

工程名稱：南方澳大橋鋼絞線拉伸試驗

委託單位：財團法人國家實驗研究院國家地震工程研究中心

聯絡資訊：N/A

承包商：N/A

供應廠商：N/A

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## 鋼絞線拉伸試驗報告

報告編號：20-00103Y

試驗日期：109.01.21

試驗機型：SHIMADZU 2000kNIR

 材料稱呼：ASTM A416/A416M(2017a) Grade 270  $\phi$ 15.7 mm 鍍鋅鋼絞線

試驗方法：ASTM A416/A416M(2017a) 及 ASTM A1061/1061M(2016)

## 一、尺度測定

試樣編號	絞合外徑 (mm)	心線線徑 (mm)	外層單線線徑 (mm)	心線與外層單線 線徑差 ( $\mu$ m)
OS8-1	15.71	5.32	5.11~5.15	0.17~0.21
OS8-2	15.73	5.31	5.11~5.14	0.17~0.20
OS8-3	15.74	5.32	5.13~5.15	0.17~0.19
OS9-1	15.73	5.32	5.12~5.16	0.16~0.20
OS9-2	15.73	5.32	5.12~5.14	0.18~0.20
OS9-3	15.73	5.31	5.13~5.15	0.16~0.18
OS9-4	15.42	5.20	5.02~5.03	0.17~0.18
OS9-5	15.40	5.20	5.02~5.06	0.14~0.18
OS9-6	15.40	5.21	5.02~5.04	0.17~0.19
OS12-1	15.76	5.34	5.12~5.14	0.20~0.22
OS12-2	15.77	5.30	5.13~5.15	0.15~0.17
OS12-3	15.77	5.32	5.12~5.13	0.19~0.20
規範值	15.55-16.35	—	—	$\geq 0.102$

## 二、單位長度質量、截面積及機械性質

試樣編號	單位長度 質量 (kg/km)	標稱截面積 ( $\text{mm}^2$ )	對應1.0%永久 伸長率之負載 (kN)	拉伸負載 (kN)	降伏強度 ( $\text{N}/\text{mm}^2$ )	抗拉強度 ( $\text{N}/\text{mm}^2$ )	伸長率 (%)	彈性模數 ( $\text{kgf}/\text{cm}^2$ )
OS8-1	1166	150.0	244.5	277.1	1630	1847	6.7	$1.90 \times 10^6$
OS8-2	1166	150.0	244.8	277.3	1632	1849	6.9	$1.90 \times 10^6$
OS8-3	1165	150.0	244.4	277.3	1629	1849	7.0	$1.88 \times 10^6$
OS9-1	1173	150.0	246.8	279.5	1645	1863	7.0	$1.93 \times 10^6$
OS9-2	1172	150.0	247.8	277.8	1652	1852	5.3	$1.93 \times 10^6$
OS9-3	1174	150.0	247.2	278.9	1648	1859	5.8	$1.89 \times 10^6$
OS9-4	1106	150.0	229.1	262.0	1527	1747	6.7	$1.79 \times 10^6$
OS9-5	1106	150.0	229.0	261.8	1527	1745	6.3	$1.79 \times 10^6$
OS9-6	1105	150.0	227.8	261.1	1519	1741	5.6	$1.78 \times 10^6$
OS12-1	1173	150.0	245.8	278.3	1639	1855	7.0	$1.91 \times 10^6$
OS12-2	1171	150.0	246.7	278.4	1645	1856	6.8	$1.94 \times 10^6$
OS12-3	1171	150.0	245.8	277.4	1639	1849	6.8	$1.89 \times 10^6$
規範值	—	—	$\geq 251.4$	$\geq 279.0$	—	—	$\geq 3.5$	—

 備註：本報告之降伏強度、抗拉強度及彈性模數係依其標稱截面積150.0  $\text{mm}^2$  所計算而得。

試驗室主管：

報告簽署人：

NOVA



華光工程顧問股份有限公司  
CECI Nova Technology Co., Ltd.



Civil Engineering  
Laboratory  
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鋼絞線試驗報告

報告編號：20-00101Y

收件日期：109.01.21

報告日期：109.01.30

工程名稱：南方澳大橋鋼絞線拉伸試驗(舊錨頭用鋼絞線)

委託單位：財團法人國家實驗研究院國家地震工程研究中心

聯絡資訊：N/A

承包商：N/A

供應廠商：N/A

取樣者：國震中心：

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試驗室主管：

報告簽署人：



華光工程顧問股份有限公司  
CECI Nova Technology Co., Ltd.



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鋼絞線拉伸試驗報告

報告編號：20-00101Y

試驗日期：109.01.21

試驗機型：SHIMADZU 2000kNIR

材料稱呼：ASTM A416/A416M(2017a) Grade 270  $\phi$ 15.7 mm 鍍鋅鋼絞線

試驗方法：ASTM A416/A416M(2017a) 及 ASTM A1061/1061M(2016)

一、尺度測定

試樣編號	絞合外徑 (mm)	心線線徑 (mm)	外層單線線徑 (mm)	心線與外層單線 線徑差 (mm)
OS10-1	15.77	5.31	5.13-5.15	0.16-0.18
OS10-2	15.78	5.32	5.14-5.15	0.17-0.18
OS10-3	15.77	5.31	5.12-5.15	0.16-0.19
以下空白				
規範值	15.55-16.35	—	—	$\geq 0.102$

二、單位長度質量、截面積及機械性質

試樣編號	單位長度 質量 (kg/km)	標稱截面積 (mm <sup>2</sup> )	對應1.0%永久 伸長率之負載 (kN)	拉伸負載 (kN)	降伏強度 (N/mm <sup>2</sup> )	抗拉強度 (N/mm <sup>2</sup> )	伸長率 (%)	彈性模數 (kgf/cm <sup>2</sup> )
OS10-1	1176	150.0	243.1	277.3	1621	1849	6.7	$1.92 \times 10^6$
OS10-2	1177	150.0	241.6	274.8	1611	1832	5.6	$1.91 \times 10^6$
OS10-3	1176	150.0	243.3	276.5	1622	1843	6.4	$1.94 \times 10^6$
以下空白								
規範值	—	—	$\geq 251.4$	$\geq 279.0$	—	—	$\geq 3.5$	—

備註：本報告之降伏強度、抗拉強度及彈性模數係依其標稱截面積150.0 mm<sup>2</sup>所計算而得。

試驗室主管：

報告簽署人：



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頁次：第1頁共2頁

鋼絞線試驗報告

報告編號：20-00102Y

收件日期：109.01.21

報告日期：109.01.30

工程名稱：南方澳大橋鋼絞線(新錨頭用鋼絞線)

委託單位：財團法人國家實驗研究院國家地震工程研究中心

聯絡資訊：N/A

承包商：N/A

供應廠商：N/A

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**鋼絞線拉伸試驗報告**

報告編號：20-00102Y

試驗日期：109.01.21

試驗機型：SHIMADZU 2000kNIR

材料稱呼：ASTM A416/A416M(2017a) Grade 270  $\phi$ 15.7 mm 鍍鋅鋼絞線

試驗方法：ASTM A416/A416M(2017a) 及 ASTM A1061/1061M(2016)

一、尺度測定

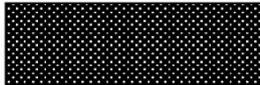
試樣編號	絞合外徑 (mm)	心線線徑 (mm)	外層單線線徑 (mm)	心線與外層單線 線徑差 (mm)
NSI-1	15.77	5.35	5.11~5.18	0.17~0.24
NSI-2	15.76	5.33	5.13~5.16	0.17~0.20
以下空白				
規範值	15.55-16.35	—	—	$\geq 0.102$

二、單位長度質量、截面積及機械性質

試樣編號	單位長度 質量 (kg/km)	標稱截面積 (mm <sup>2</sup> )	對應1.03永久 伸長率之負載 (kN)	拉伸負載 (kN)	降伏強度 (N/mm <sup>2</sup> )	抗拉強度 (N/mm <sup>2</sup> )	伸長率 (%)	彈性模數 (kgf/cm <sup>2</sup> )
NSI-1	1197	150.0	250.6	290.9	1671	1939	6.4	$1.90 \times 10^6$
NSI-2	1196	150.0	251.5	290.3	1677	1935	5.8	$1.92 \times 10^6$
以下空白								
規範值	—	—	$\geq 251.4$	$\geq 279.0$	—	—	$\geq 3.5$	—

備註：本報告之降伏強度、抗拉強度及彈性模數係依其標稱截面積150.0 mm<sup>2</sup>所計算而得。

試驗室主管：



報告簽署人：



附錄 4 材料試驗報告

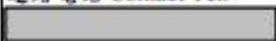
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 Materials & Electro - Optics Research Division, National Chung Shan Institute of Science and Technology  
 桃園龍潭郵政 90008-8-5 信箱 P.O. Box 90008-8-5, Lungtan, Taoyuan 32500, Taiwan, R.O.C.  
 TEL: (03)-4711742, FAX: (03)-4714368

材料測試報告

編號(No.): 109 專-7-043

Materials Test Report

頁次(page): 1

申請單位名稱和地址 Name and Address of Client 國家運輸安全調查委員會		來文編號 Application No. 無	
試樣名稱 Name of Sample 南方澳斷橋部件		項數 / 件數 No. of Items/Pieces /	
試驗項目 Test Items 表面缺陷觀察、成份分析、硬度測試、拉伸試驗、 鍍層分析、金相觀察等		日期 Date Y/M/D	建案 Acceptance 109/03/05
試驗方法 (規範) Test Methods/Specifications 詳如內文		完成 Issue 109/04/27	
		試驗儀器 Test Instruments OM、SEM/EDS、SPARK-OES、碳硫分析儀、 EPMA、維氏硬度機、拉伸試驗機、CT 電腦 斷層掃描、螢光檢驗探傷裝置	
<p><b>一、試樣說明</b></p> <p>本案為國家運輸安全調查委員會(後文簡稱運安會)委託,針對 108 年 10 月 1 日南方澳斷橋事件之相關部件進行分析,分析部件主要包括錨頭、鋼絞線、鋼線、楔形夾片等,檢測項目則包含材質分析、機械性質(硬度及強度)測試、破断面觀察及破断面形態分析、表面鍍層分析及金相觀察等,以及螢光檢驗探傷與 CT 電腦斷層掃描等非破壞性檢驗,其中鋼絞線及錨頭連帶鋼絞線的拉伸試驗由運安會另委託國家地震工程研究中心(後文簡稱國震中心)進行。</p> <p>本案取得之待測試樣包含編號 8 號吊索之鋼絞線 2 束、9 號鋼絞線 13 束及 12 號鋼絞線 13 束,上拱錨頭編號 1~13 號及橋面端錨頭編號 1~13 號。其中鋼線的部份分別自上拱端取斷裂處上方適當長度以進行破断面觀察及前述各項分析。而上拱錨頭的部份,編號 10 及 12 號錨頭所有鋼絞線仍留存在錨頭上,故用以進行錨頭/鋼絞線之拉伸測試,其餘上拱錨頭若有部份鋼絞線殘留,則先委請國震中心退除後再交由本院進行後續分析(其中編號 7、9、11 號上拱錨頭未破裂,另進行 CT 檢測及孔位量測)。橋面端錨頭僅 1 號破裂,可直接取樣分析,其餘橋面錨頭因仍連結承壓環及螺紋調整管,並有部份鋼絞線殘留,故亦委請國震中心進行拆解及退除鋼絞線後再交由本院分析。</p>			
填寫 Prepared by		審查 Reviewed by	
連絡電話 Contact Tel. 		核定(報告簽署人) Approved by	

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# 材料測試報告

## Materials Test Report

編號(No.)：109 專-7-043

頁次(page)： 2/156

### 二、編號 10 及 12 號上拱錨頭螢檢探傷

由於上拱錨頭 10 及 12 號之 13 束鋼絞線皆完整留存固定於錨頭上，故可用於進行錨頭連帶鋼絞線之拉伸測試以釐清錨頭與鋼絞線兩者之強弱關係。在進行拉伸測試之前，為避免錨頭本身表面或內部已有裂隙影響實驗結果，需先對錨頭進行非破壞性檢測以了解是否有裂隙存在，但由與錨頭連帶鋼絞線之整體尺寸過大，無法以本院之 X 光電腦斷層(CT)設備進行檢測，故改以螢光檢驗探傷方式進行檢驗。

圖 1 為上拱錨頭 10 號及 12 號連同鋼絞線之外觀，兩錨頭之上表面因未被帽蓋保護，皆有較明顯之生鏽現象。在進行螢光檢驗前，先去除表面髒污及油污，避免影響判斷，但由於錨頭心部為 13 束鋼絞線貫穿之孔位，不易清理，故螢光檢驗主要針對側面及上下表面外圍處進行。上拱 10 號錨頭側面共發現兩處有表面裂紋，皆位於側面螺紋之齒峰位置，如圖 2 中紅色方框標示處，圖中 A 處裂紋有向下延伸，如圖 3 所示，但目視判斷其深度較淺約至接近齒底處。而 B 處裂紋位於連續三齒之齒峰位置，且紋路筆直，疑似表面刮傷造成。錨頭上下表面之外圍則未發現明顯裂紋存在，如圖 4 所示。上拱 12 號錨頭未被套筒保護之上表面有較多鏽斑，側面亦可觀察到鏽斑，如圖 5 所示。螢光檢驗發現側面亦有 2 處有表面裂縫，如圖 6 所示，但亦屬位於齒峰之筆直紋路，且深度很淺；上下表面則無發現明顯裂紋。由螢光檢驗探傷結果可知，上拱 10 號及 12 號兩個錨頭在側面皆發現兩處疑似表面裂紋，但目視判斷其深度皆相當淺，研判不會對後續拉伸試驗有重大影響。



圖1 上拱錨頭 10 號及 12 號連同鋼絞線之外觀

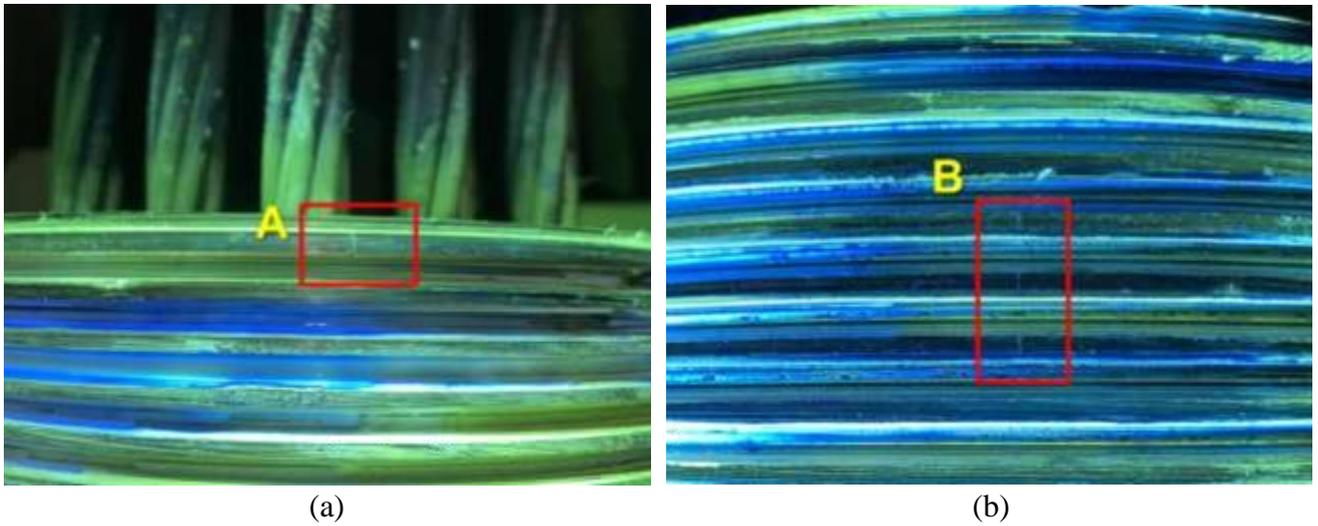


圖2 上拱錨頭 10 號側面螺牙處之表面裂紋

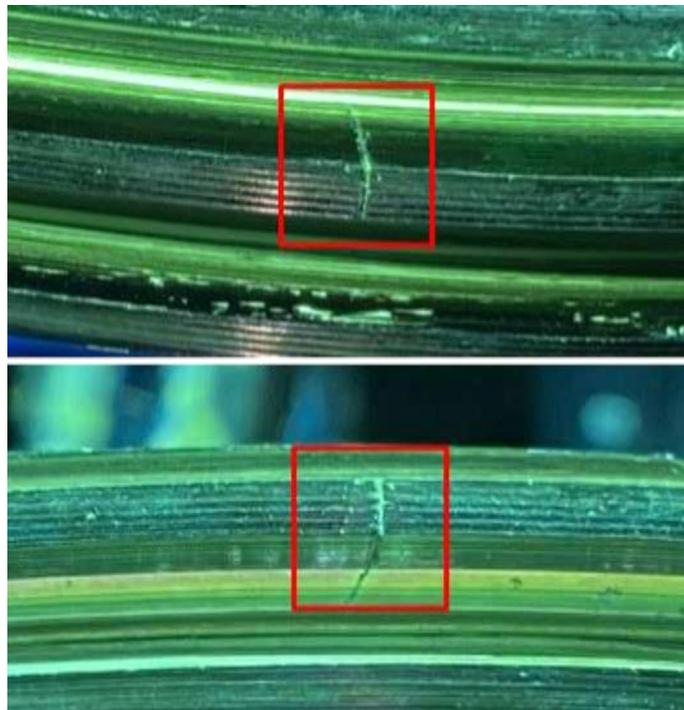


圖3 上拱錨頭 10 號 A 處表面裂紋放大觀察照片

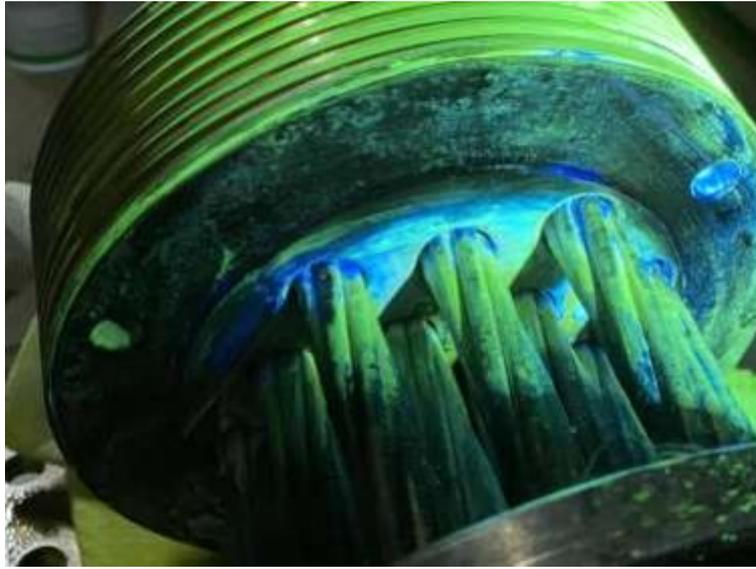
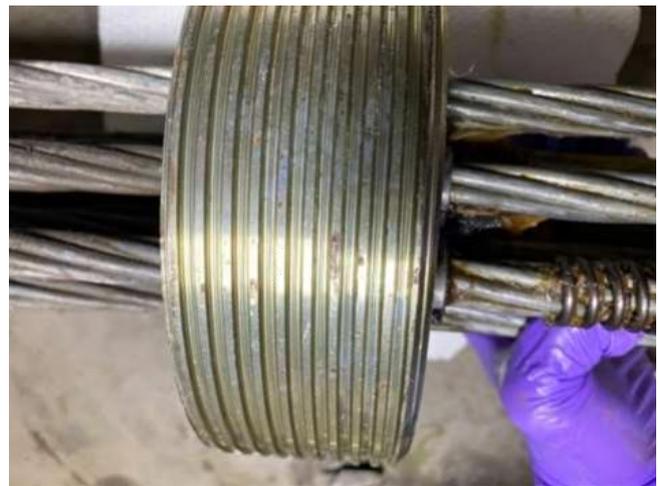


圖4 上拱錨頭 10 號上表面之螢光檢驗照片

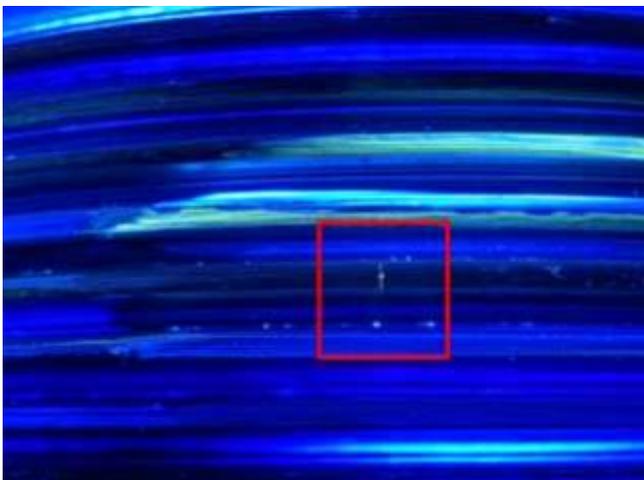


(a)

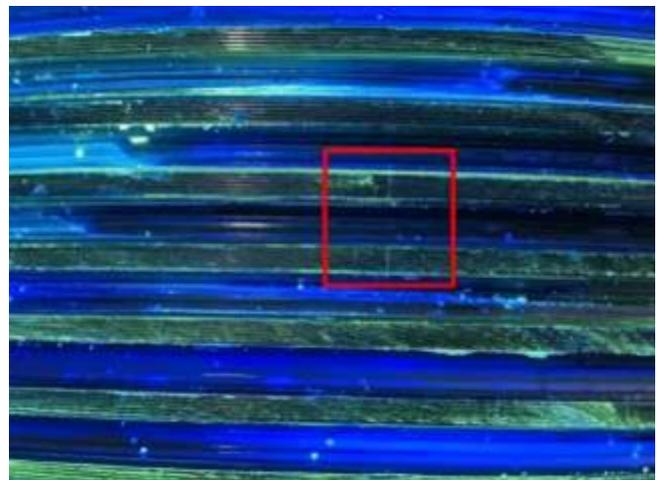


(b)

圖5 上拱錨頭 12 號外觀



(a)



(b)

圖6 上拱錨頭 12 號側面螺牙處之表面裂紋

### 三、編號 7、9 及 11 號上拱錨頭 CT 檢驗

依國震中心之規劃，除了上拱錨頭 10 及 12 號可進行錨頭連帶鋼絞線之拉伸測試外，另上拱錨頭 7、9 及 11 號未破裂，可搭配新製鋼絞線進行拉伸測試以了解錨頭強度，同樣在進行拉伸測試之前，需先對錨頭進行非破壞性檢測以了解是否有裂隙存在，此三件錨頭留存之鋼絞線已退除，可使用本院 X 光電腦斷層(CT)進行檢測分析內缺陷。

圖 7~圖 9 為三件錨頭外觀照片，錨頭之上表面皆有鏽蝕現象，特別是 9 號及 11 號鏽蝕更嚴重，而下表面因有帽蓋保護，氧化程度相對較輕微，仍可明顯觀察到表面鍍層。在進行 CT 檢驗之前先以目視檢查方式檢驗錨頭表面是否有明顯傷痕或裂紋，此外，為避免表面髒污影響檢測結果，在 CT 檢驗之前亦先以酒精去除工件表面之油污後再進行檢測。圖 10 為上拱錨頭 7 號之外觀傷痕檢視照片，於側面之螺紋齒峰處可觀察到表面受撞擊之凹痕，如圖 10(a)(b)中箭頭所指處，但目視判斷其凹痕深度較淺；另於錨頭之螺孔處亦發現有受到撞擊造成之變形痕跡存在，如圖 10(c)(d)中箭頭所指處。圖 11 為上拱錨頭 7 號之 CT 檢驗圖片，其內部並無發現明顯裂紋存在。圖 12 為上拱錨頭 9 號之外觀傷痕檢視照片，於上表面與側面交接處之第一階螺紋，發現有螺牙變形斷裂位移之現象，且左右兩邊皆有發現，如圖 12(a)(b)所示；由上表面相對位置觀察可發現有相對應之變形壓痕存在，如圖 12(c)(d)所示，此變形導致下方螺牙變形斷裂，由其形貌判斷為圓形狀物體壓迫或撞擊所導致變形。圖 13 為上拱錨頭 9 號之 CT 檢驗圖片，其內部並無發現明顯裂紋存在。圖 14 為上拱錨頭 11 號之外觀傷痕檢視照片，於側面螺牙發現一處有裂縫存在，如圖 14(a)(b)中箭頭所指處（二圖為同一道裂紋），此外於側面之多處齒峰位置，也發現有許多凹痕或撞擊之痕跡，如圖 14(c)(d)(e)中箭頭所指處；圖 15 為上拱錨頭 11 號之 CT 檢驗圖片，可觀察到圖 14(a)(b)中之裂縫，但其裂縫深度約到齒谷位置，並未往內部延伸，而內部亦無發現有明顯裂紋存在。



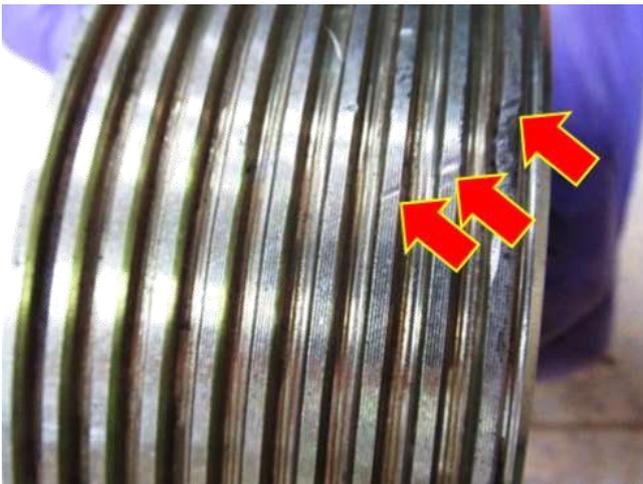
圖7 上拱錨頭 7 號外觀



圖8 上拱錨頭 9 號外觀



圖9 上拱錨頭 11 號外觀



(a)



(b)

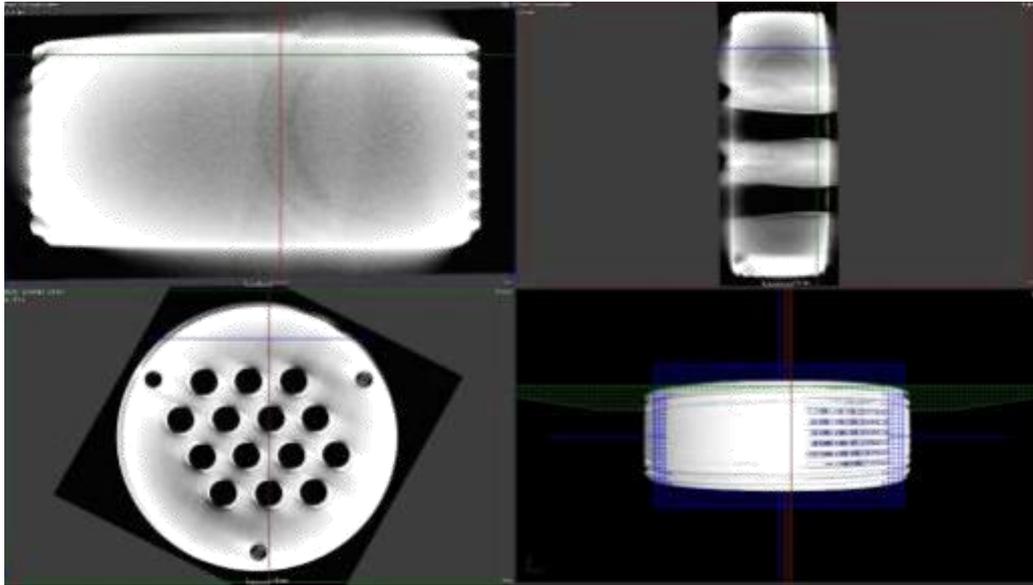


(c)

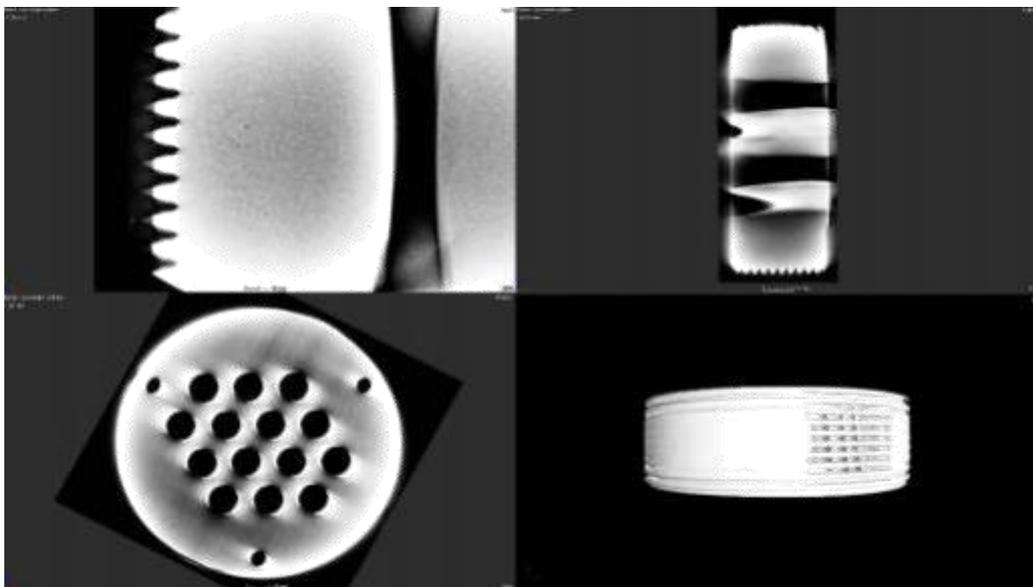


(d)

圖10 上拱錨頭 7 號外觀傷痕



(a)



(b)

圖11 上拱錨頭7號 CT 檢測圖片



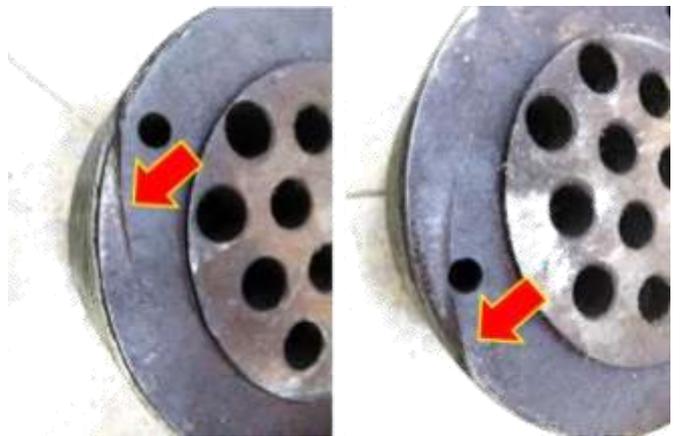
(a)



(b)



(c)



(d)

圖12 上拱錨頭 9 號外觀傷痕

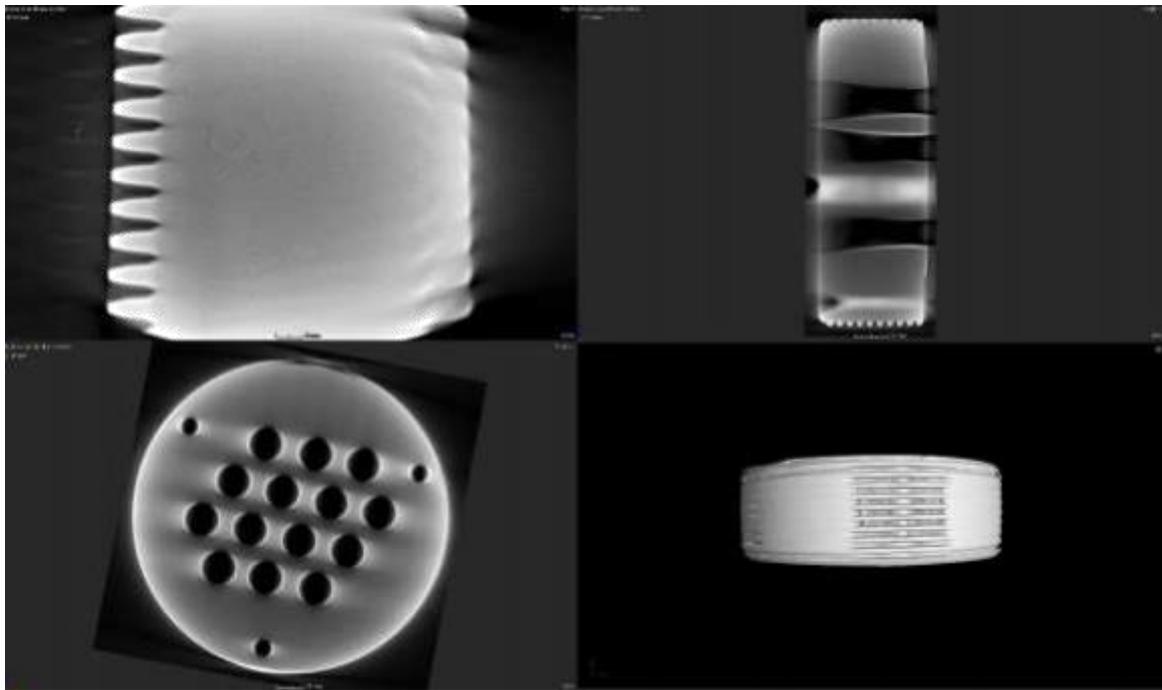
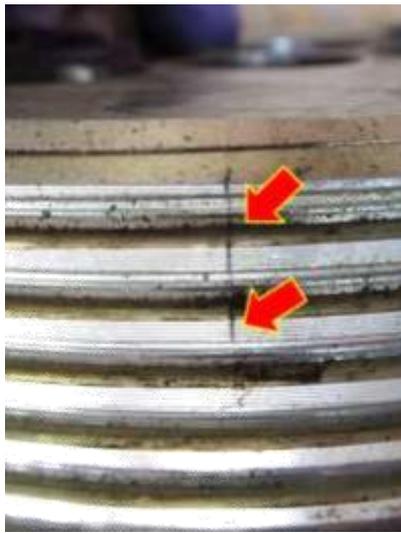


圖13 上拱錨頭 9 號 CT 檢測圖片



(a)



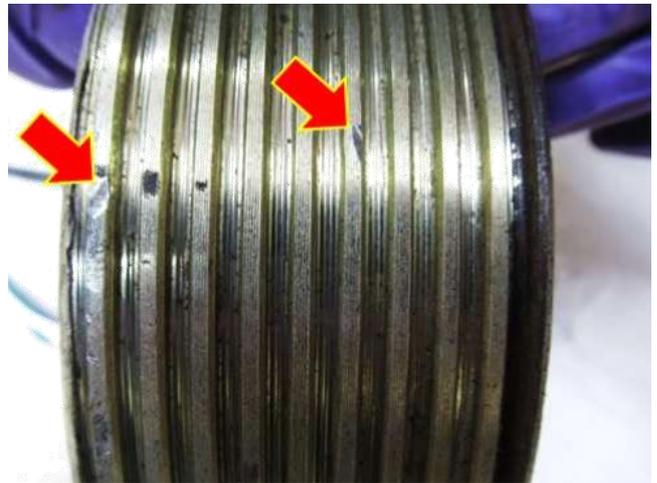
(b)



(c)



(d)



(e)

圖14 上拱錨頭 11 號外觀傷痕

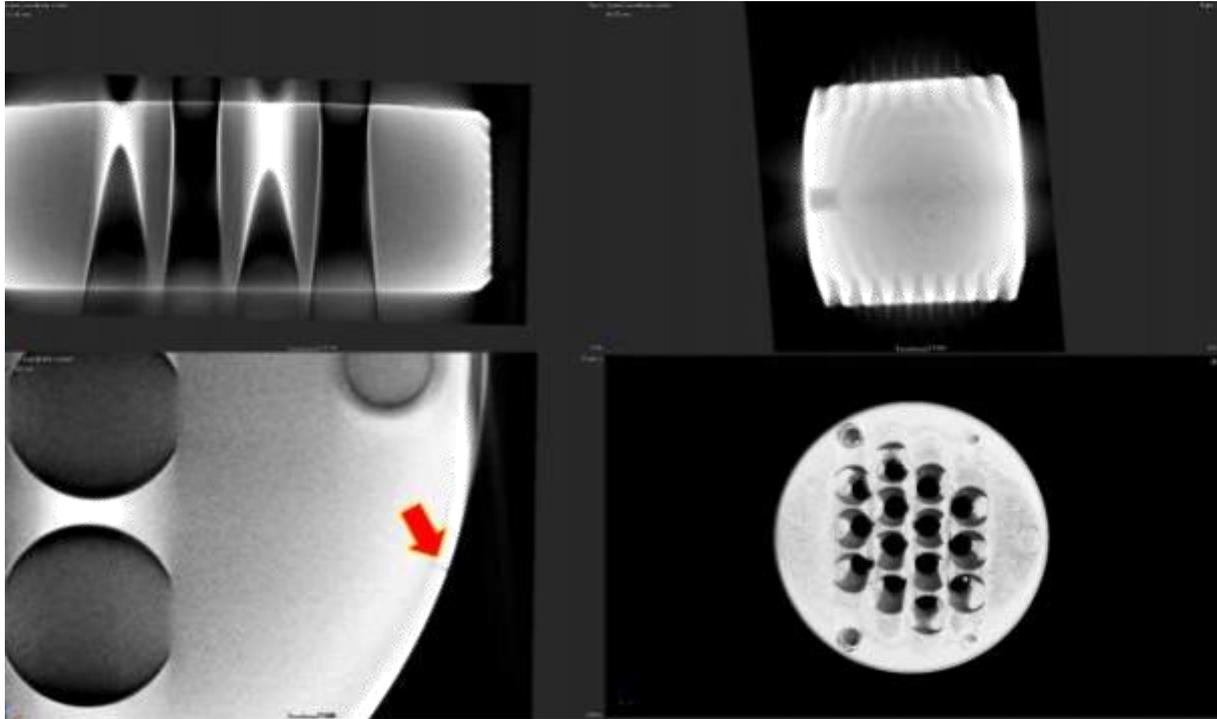


圖15 上拱錨頭 11 號 CT 檢測圖片

#### 四、錨頭成份分析

由於完整之錨頭可用於拉伸測試，故進行錨頭材料之成份分析時，選取破裂之錨頭，取樣編號為上拱錨頭 6 號及 8 號與橋面錨頭 1 號。取適當之錨頭破裂塊，經研磨後以火花發射光譜儀(SPARK-OES)進行成份分析，分析結果如表 1 所示，三件錨頭試樣之分析結果皆符合 JIS S45C 規格，此規格亦符合 AISI 1045 規格。

表1 錨頭試樣之成份分析結果

試樣名稱	元素(wt%)									
	C	Si	S	P	Mn	Ni	Cr	Mo	Cu	Fe
上拱 6 號	0.45	0.27	0.005	0.017	0.67	0.05	0.12	0.01	0.13	Rem.
上拱 8 號	0.44	0.27	0.006	0.018	0.67	0.06	0.11	0.01	0.15	Rem.
橋面 1 號	0.45	0.27	0.002	0.017	0.68	0.05	0.11	0.01	0.14	Rem.
JIS G4051 S45C 規格	0.42~ 0.48	0.15~ 0.35	0.035 Max.	0.030 Max.	0.60~ 0.90	--	0.20 Max.	--	--	Rem.
SAE-AISI 1045 規格	0.43~ 0.50		0.050 Max.	0.040 Max.	0.60~ 0.90					Rem.

### 五、上拱錨頭材質拉伸試驗

為了解錨頭使用之材質強度，規劃進行拉伸試驗。取上拱 3 號錨頭進行試驗，錨頭外觀如圖 16 所示，事故發生時已破裂，故選取肉厚較多的位置進行取樣，並加工為拉伸試棒，取樣位置及加工後與安裝後外觀如圖 17 所示，共取兩件試樣，編號為#1 及#2，拉伸試驗結果如表 2 所示。



圖 16 上拱錨頭 3 號外觀



(a)

(b)

圖 17 上拱錨頭 3 號拉伸試樣取樣位置及拉伸試棒加工後與安裝於夾具上外觀

表 2 上拱 3 號錨頭拉伸試驗結果

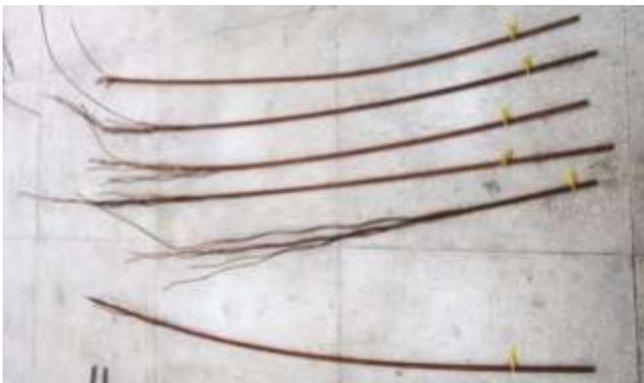
試樣編號	降伏強度(MPa)	極限抗拉強度(MPa)	伸長率(%)
上拱 3 號#1	411	700	19.8
上拱 3 號#2	347	675	22.6
平均	379	688	21.2

### 六、鋼絞線斷面觀察

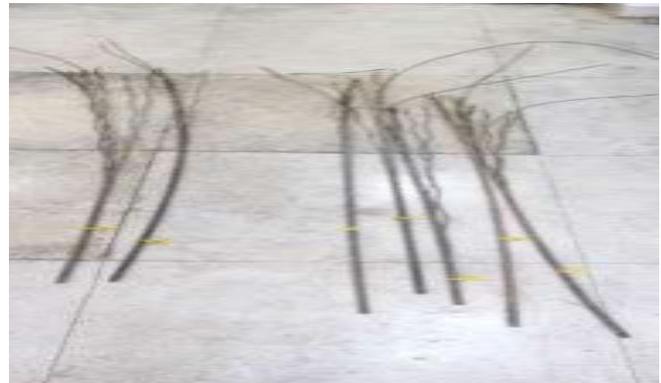
本院取得之鋼絞線取自編號 8、9、12 號吊索，外觀如圖 18～圖 20 所示，其中 9 號吊索之鋼絞線有 6 束為整段生鏽，另有 7 束則為一般氧化表面，如圖 19 所示。針對各鋼絞線束之各鋼線進行斷面觀察，以便後續以斷面形貌進行分類，各鋼線斷面之外觀照片如圖 21～圖 48 所示。



圖18 編號第 8 股吊索之 2 束鋼絞線外觀



(a)



(b)

圖19 編號第 9 股吊索之鋼絞線外觀：(a) 生鏽 6 束；(b) 未生鏽 7 束



圖20 編號第 12 股吊索之鋼絞線外觀

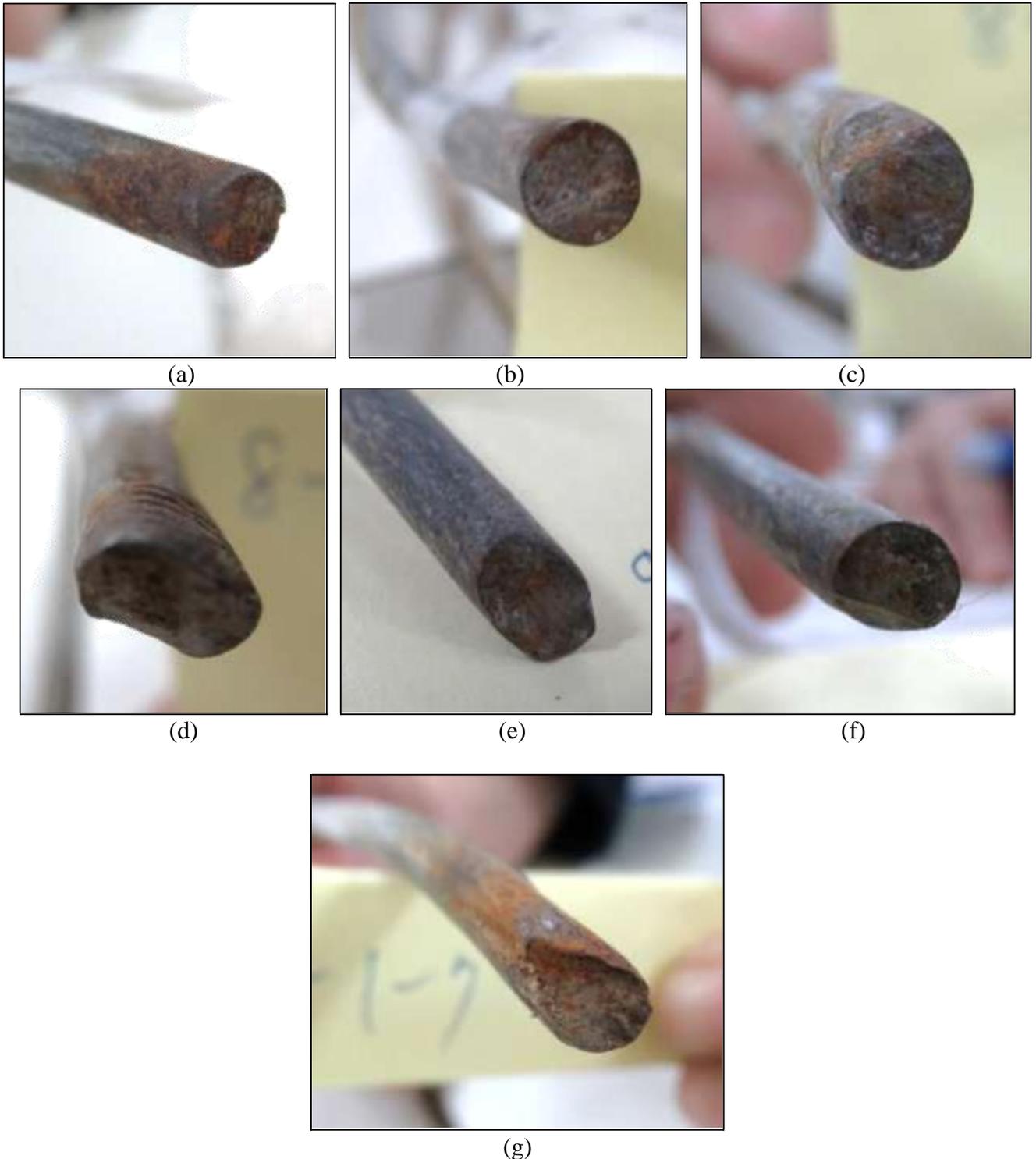


圖21 編號第 8 股吊索第 1 束鋼絞線之各鋼線斷面外觀



(a)



(b)



(c)



(d)



(e)



(f)



(g)

圖22 編號第8股吊索第2束鋼絞線之各鋼線斷面外觀



(a)



(b)



(c)



(d)



(e)



(f)



(g)

圖23 編號第9股吊索第1束鋼絞線之各鋼線斷面外觀

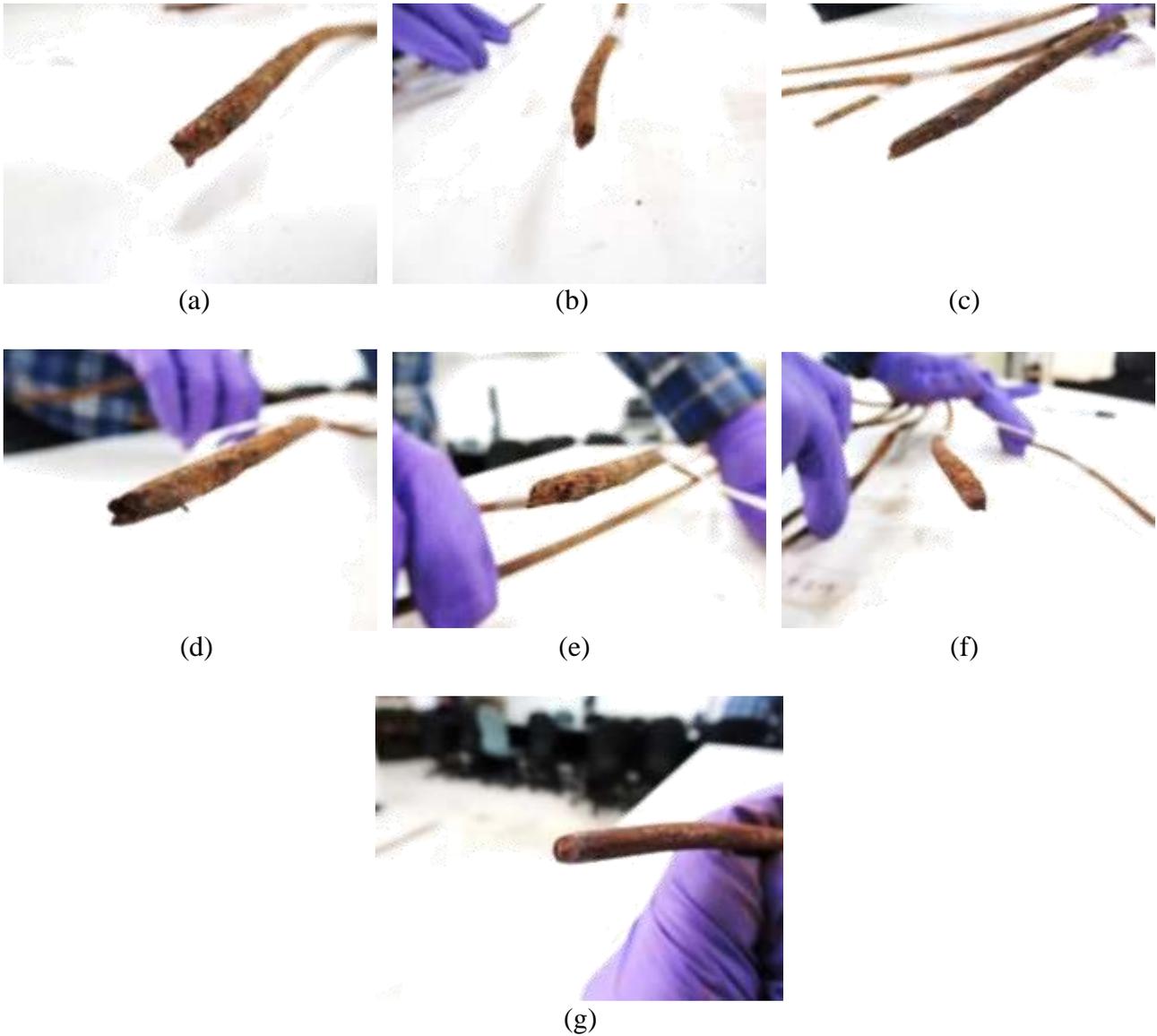


圖24 編號第9股吊索第2束鋼絞線之各鋼線斷面外觀

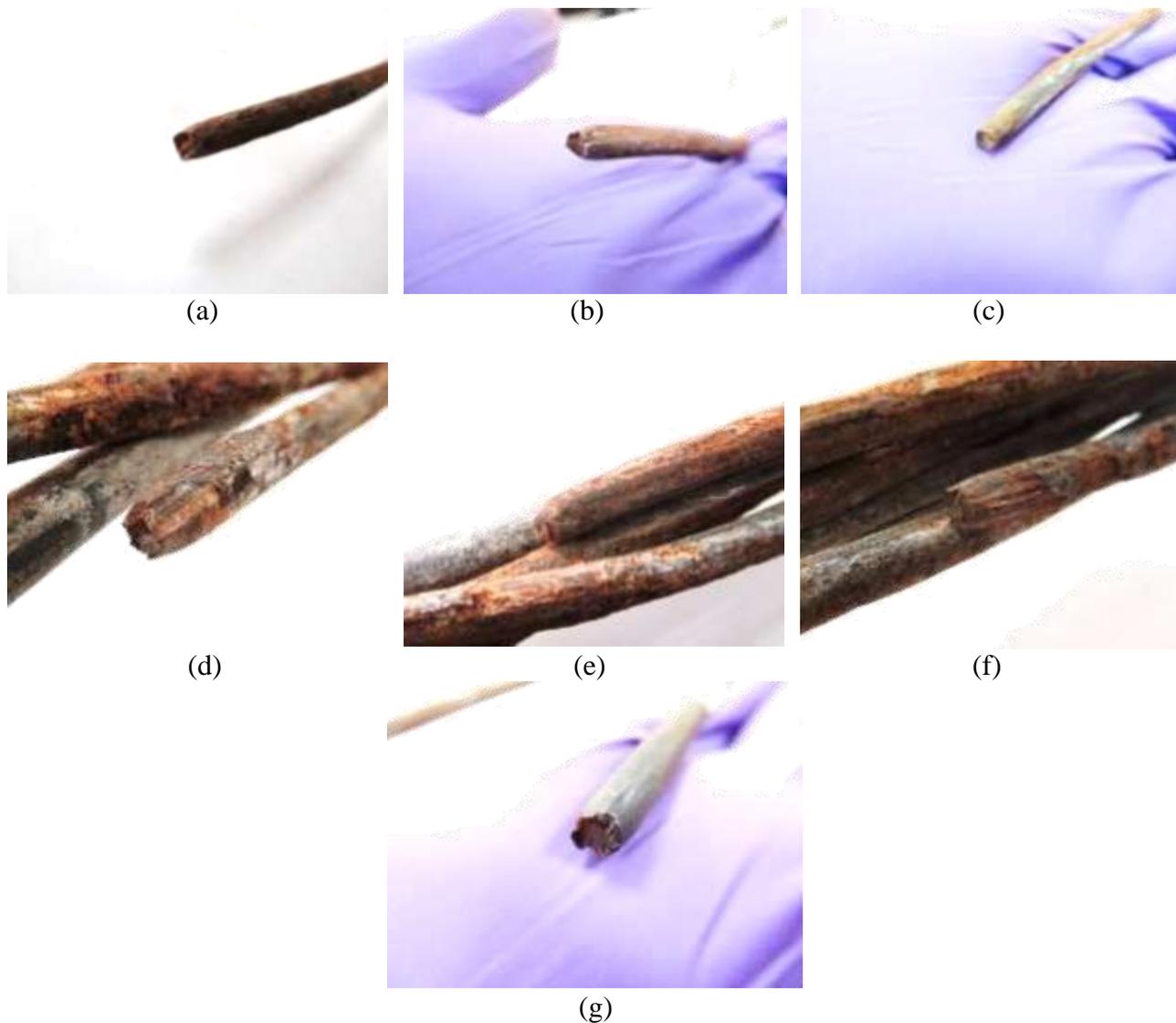


圖25 編號第9股吊索第3束鋼絞線之各鋼線斷面外觀

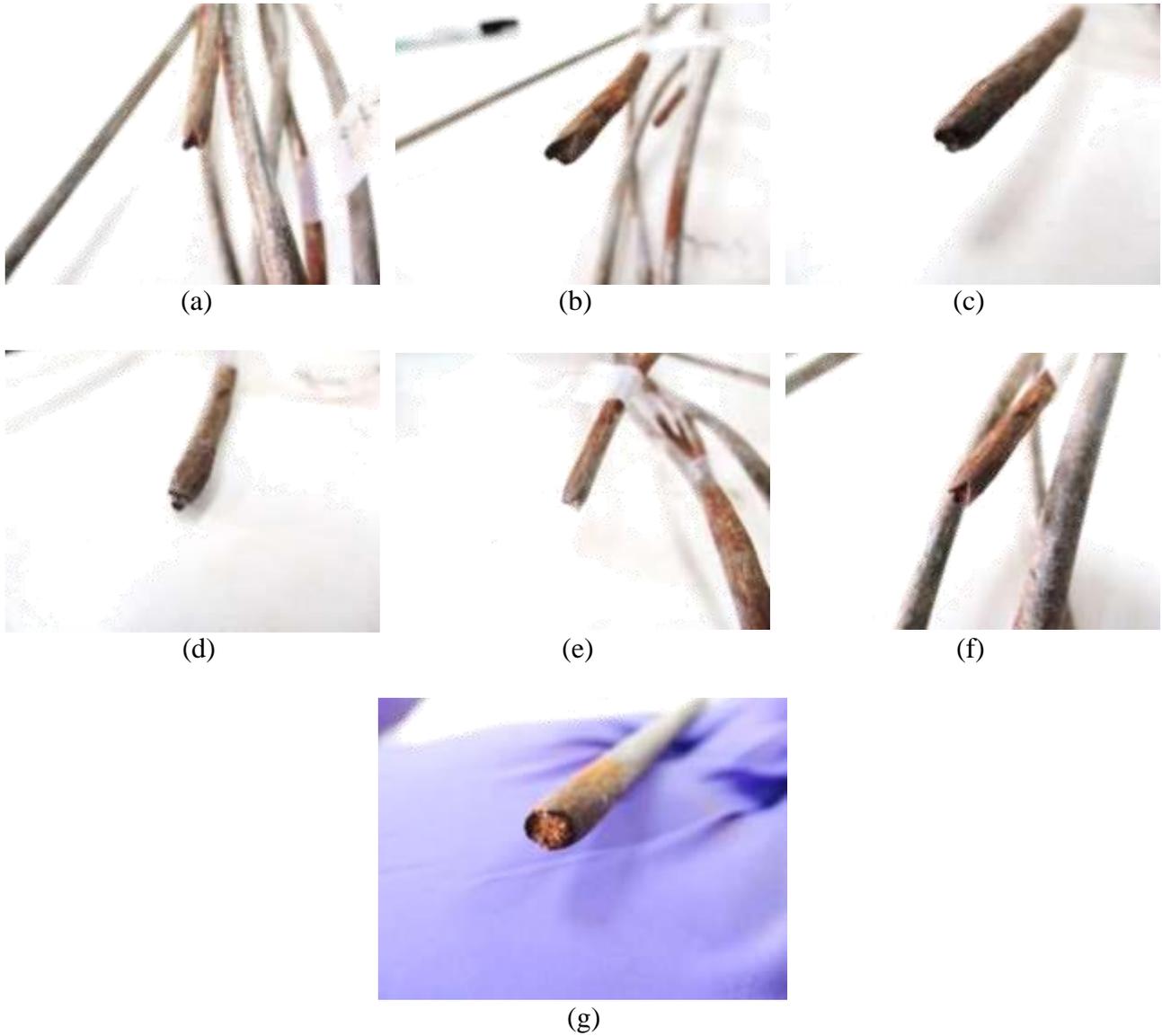


圖26 編號第9股吊索第4束鋼絞線之各鋼線斷面外觀

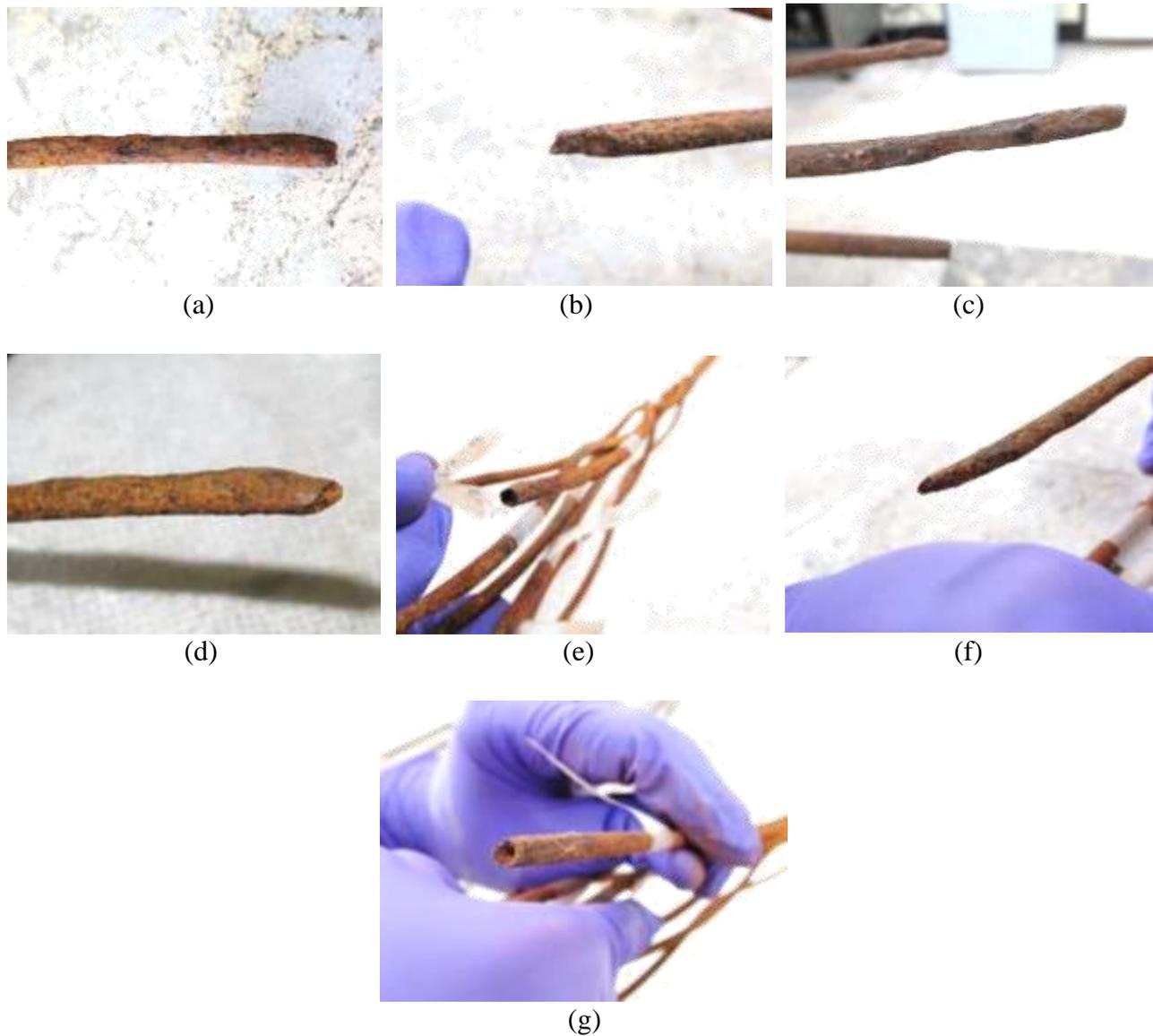


圖27 編號第 9 股吊索第 5 束鋼絞線之各鋼線斷面外觀

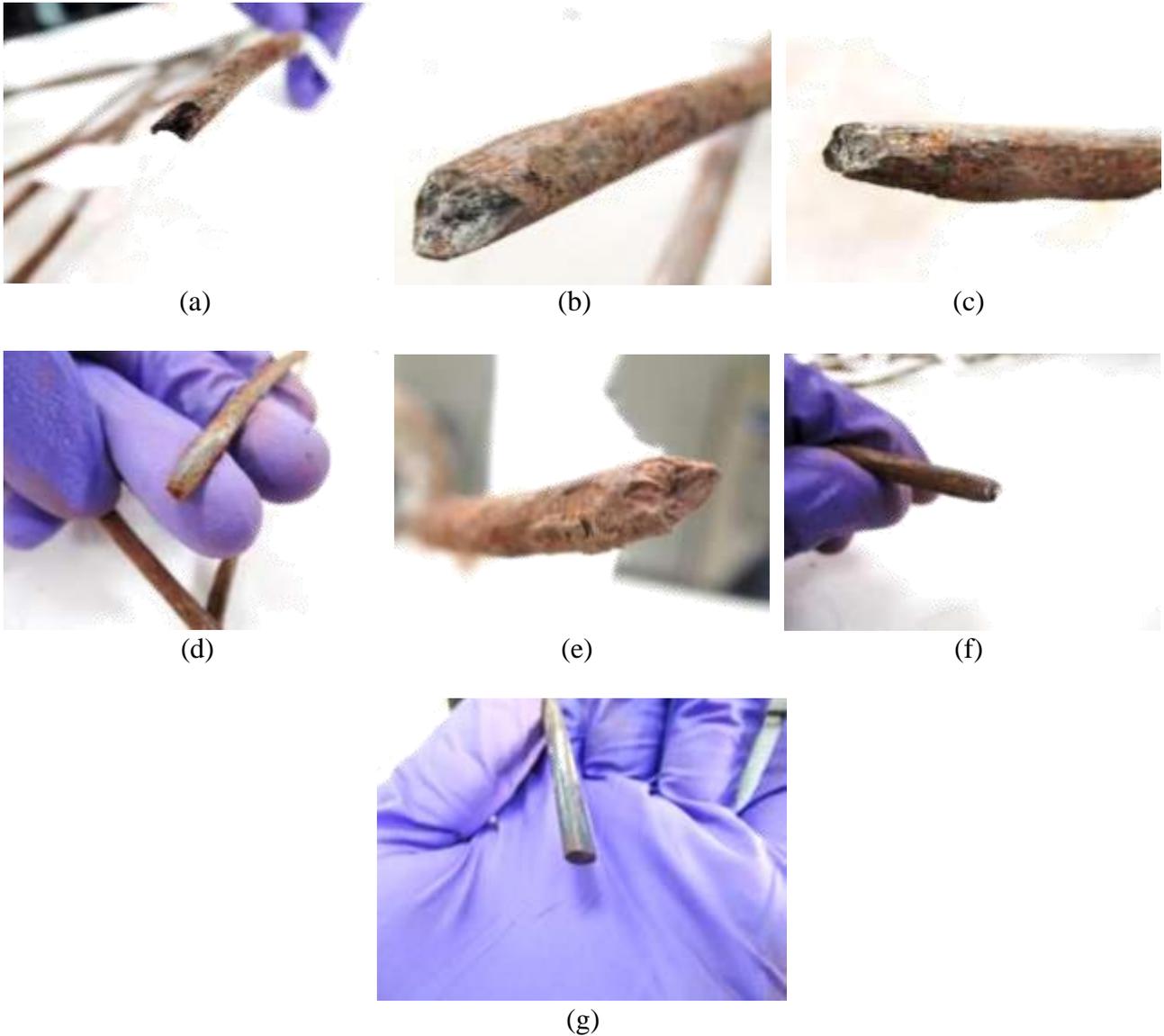


圖28 編號第9股吊索第6束鋼絞線之各鋼線斷面外觀

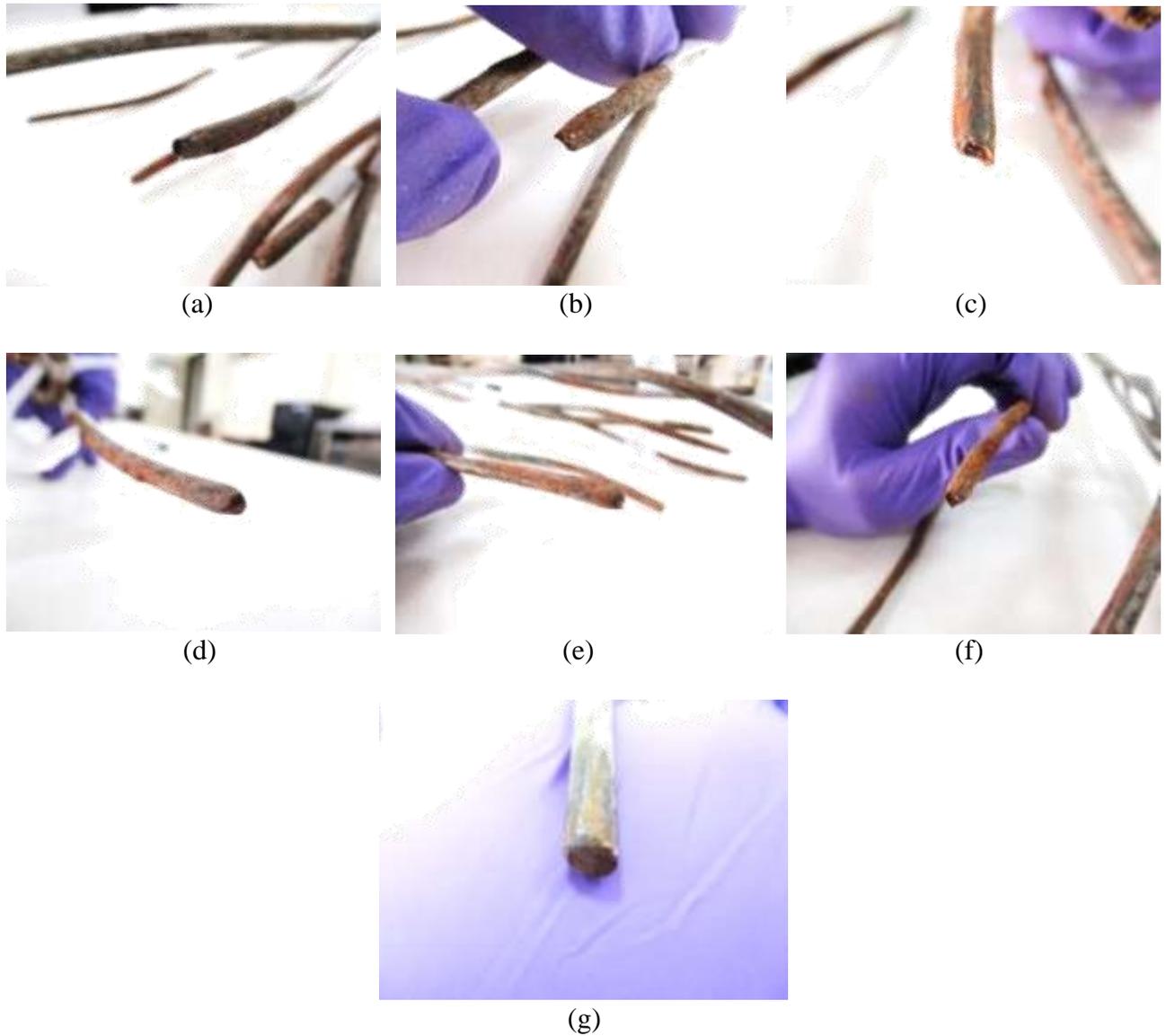


圖29 編號第 9 股吊索第 7 束鋼絞線之各鋼線斷面外觀

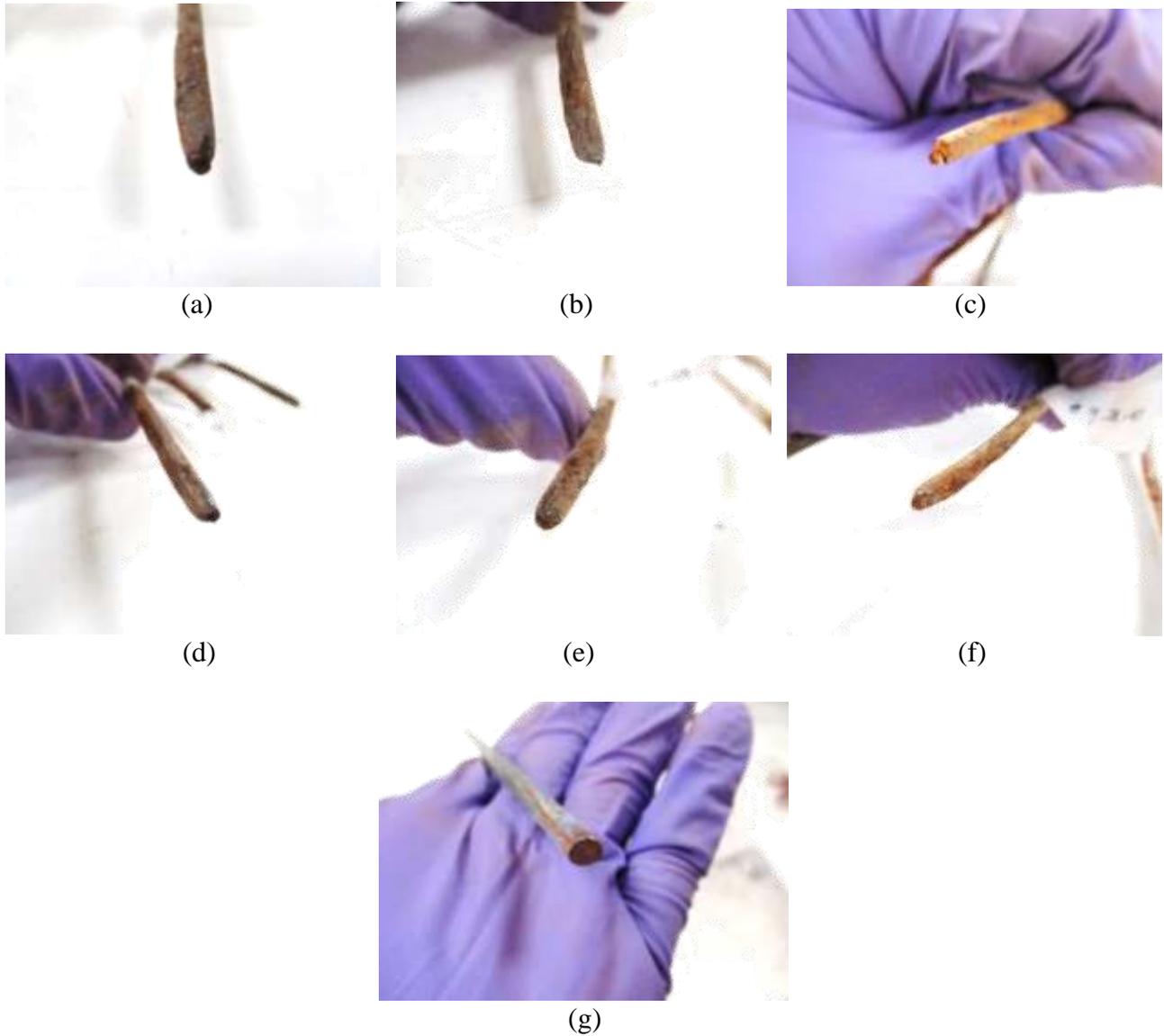


圖30 編號第9股吊索第8束鋼絞線之各鋼線斷面外觀

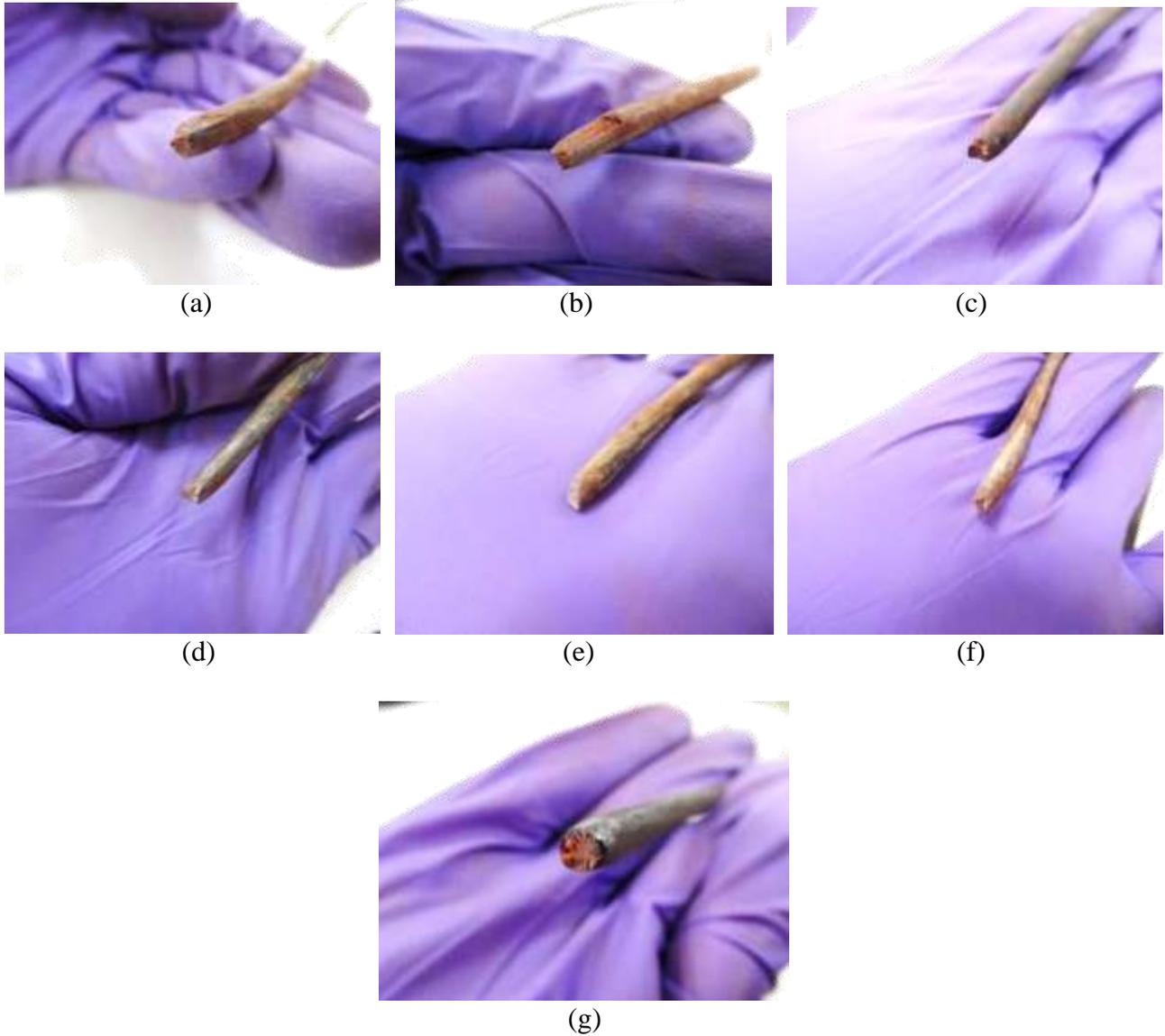


圖31 編號第 9 股吊索第 9 束鋼絞線之各鋼線斷面外觀



圖32 編號第9股吊索第10束鋼絞線之各鋼線斷面外觀

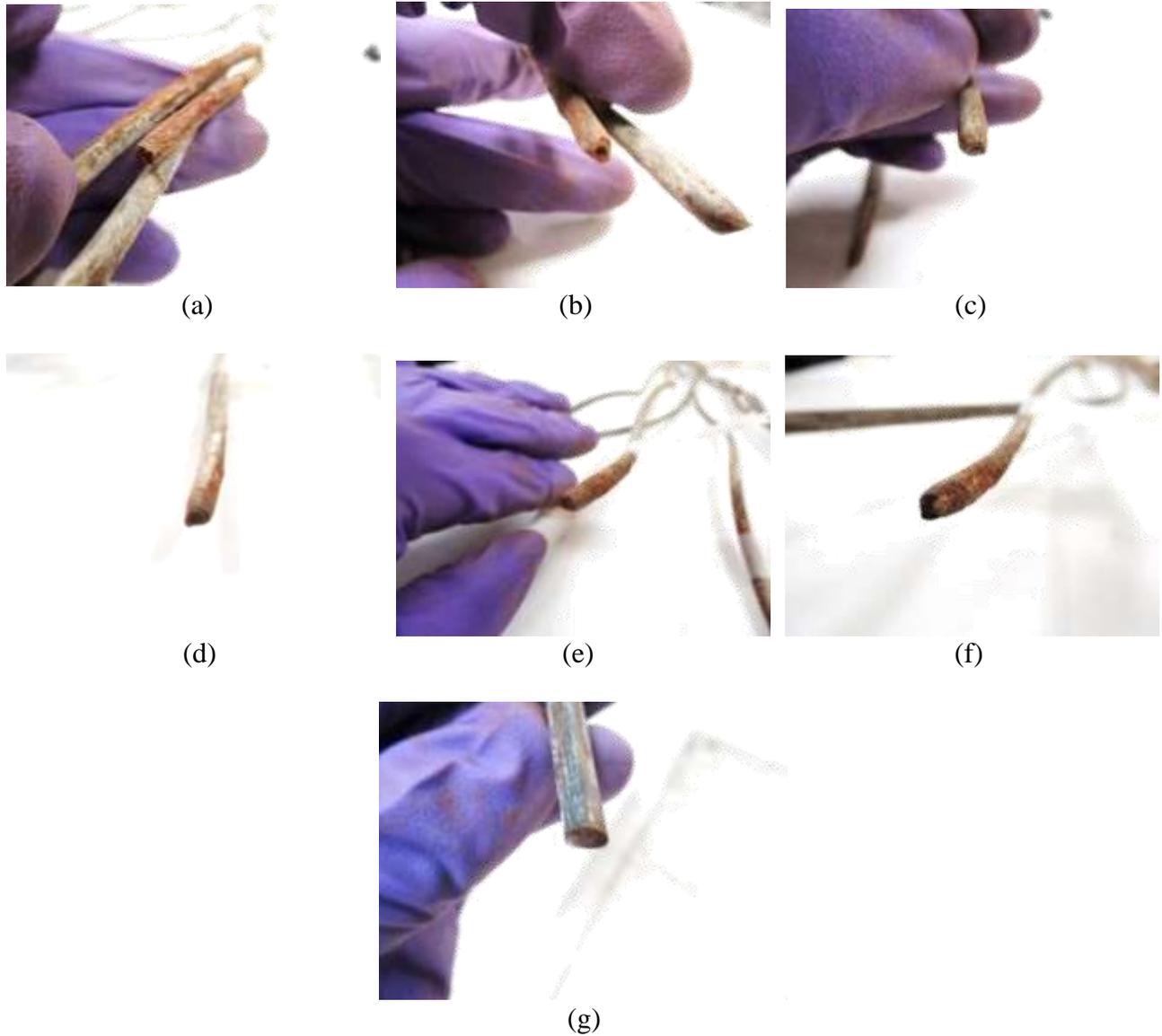


圖33 編號第9股吊索第11束鋼絞線之各鋼線斷面外觀

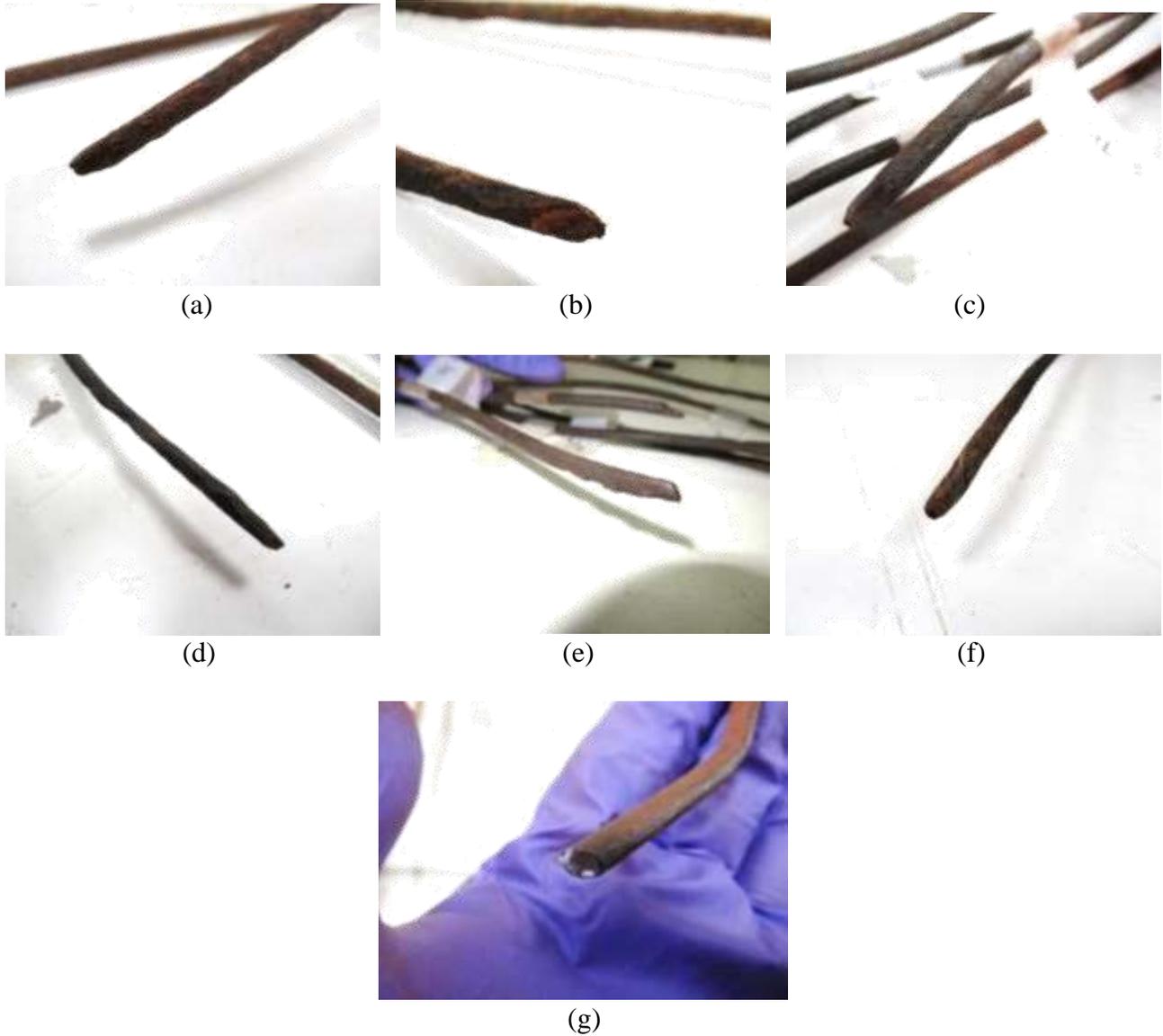


圖34 編號第9股吊索第12束鋼絞線之各鋼線斷面外觀



(a)



(b)

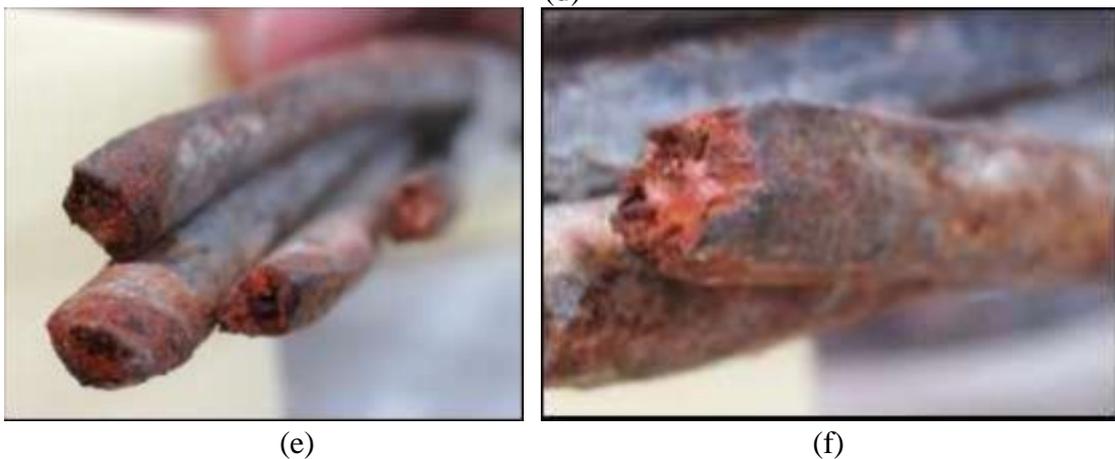
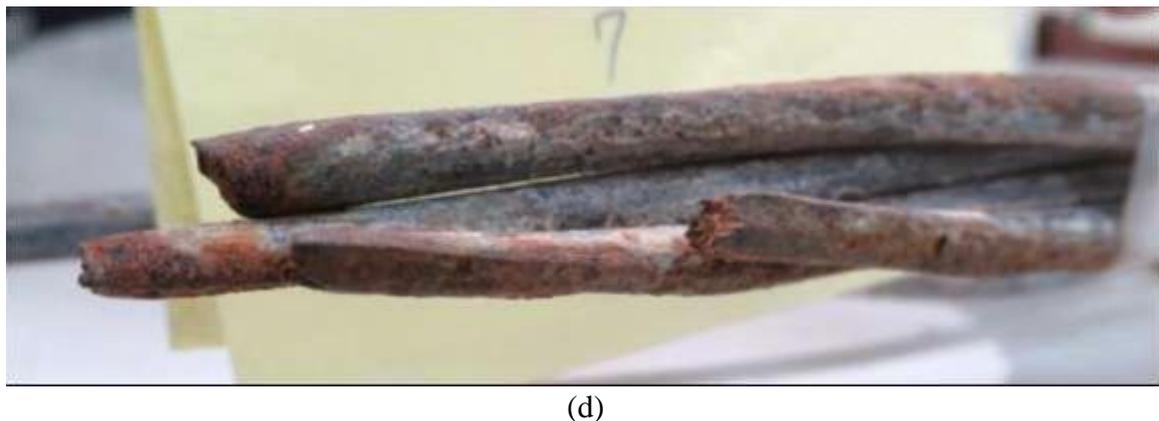
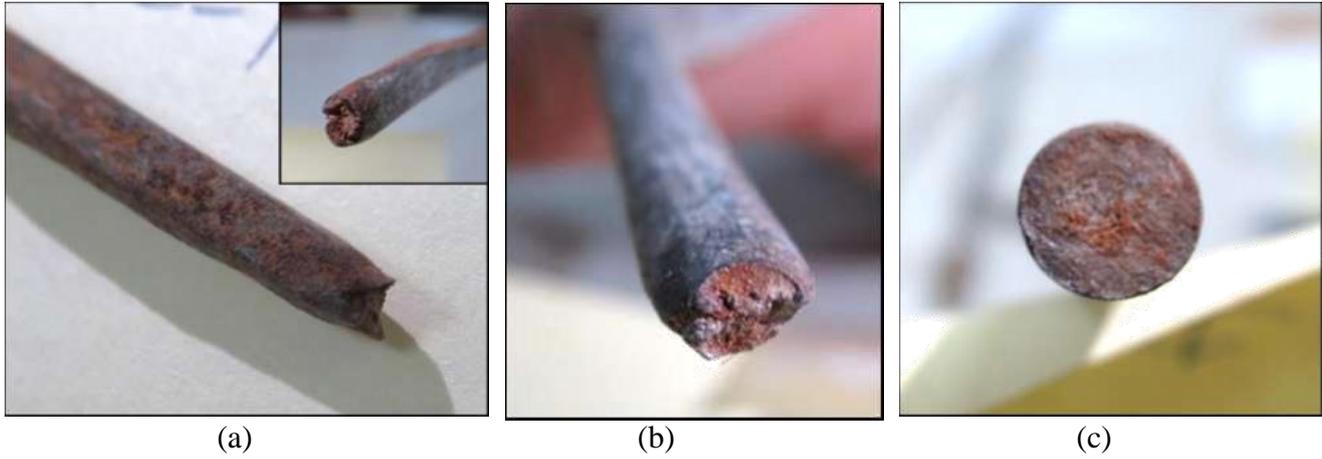


(c)



(d)

圖35 編號第9股吊索第13束鋼絞線之各鋼線斷面外觀



(e) (f)  
圖36 編號第 12 股吊索第 1 束鋼絞線之各鋼線斷面外觀



(a)



(b)



(c)



(d)



(e)



(f)



(g)

圖37 編號第 12 股吊索第 2 束鋼絞線之各鋼線斷面外觀



(a)



(b)



(c)



(d)



(e)



(f)



(g)

圖38 編號第 12 股吊索第 3 束鋼絞線之各鋼線斷面外觀

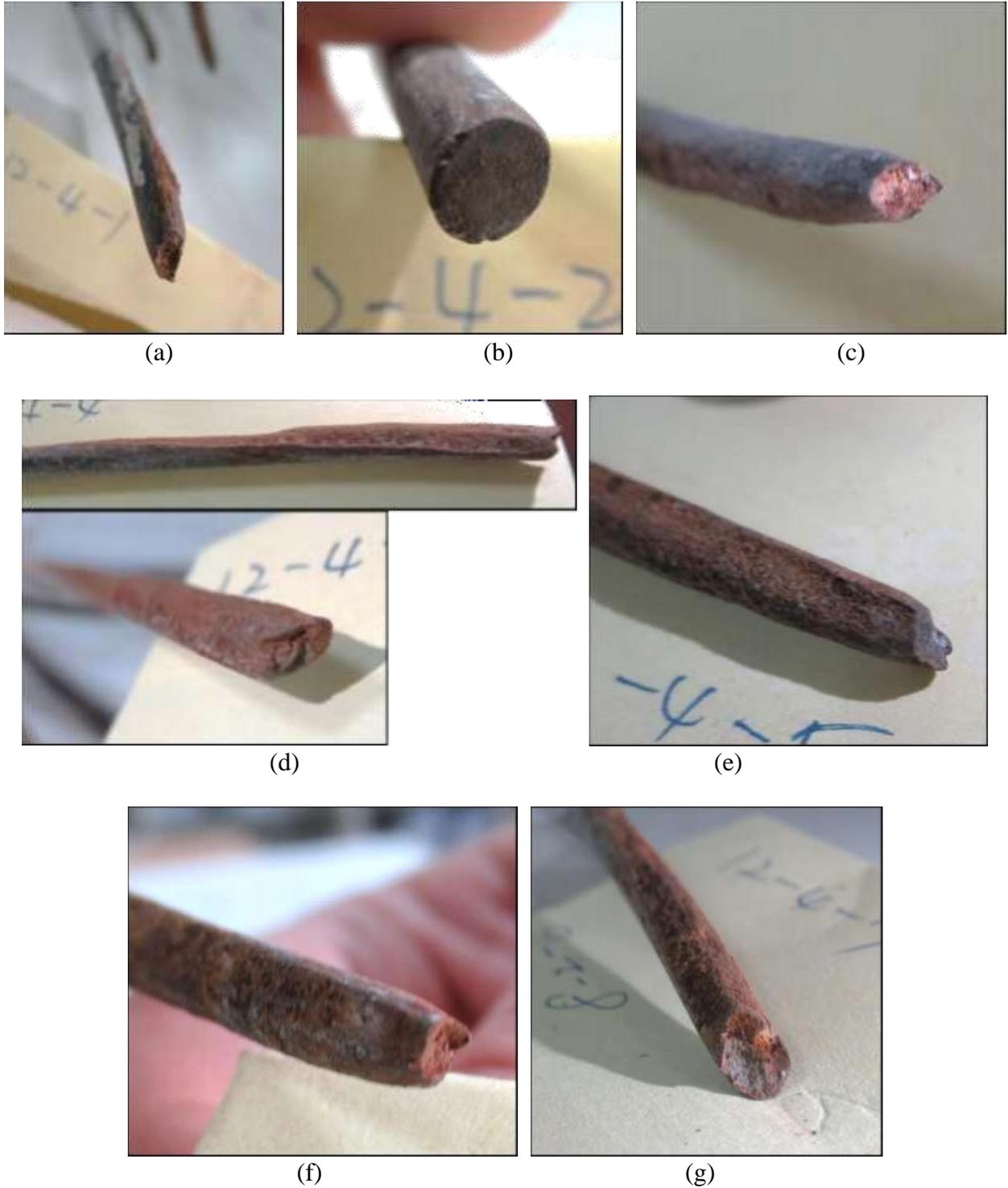


圖39 編號第 12 股吊索第 4 束鋼絞線之各鋼線斷面外觀

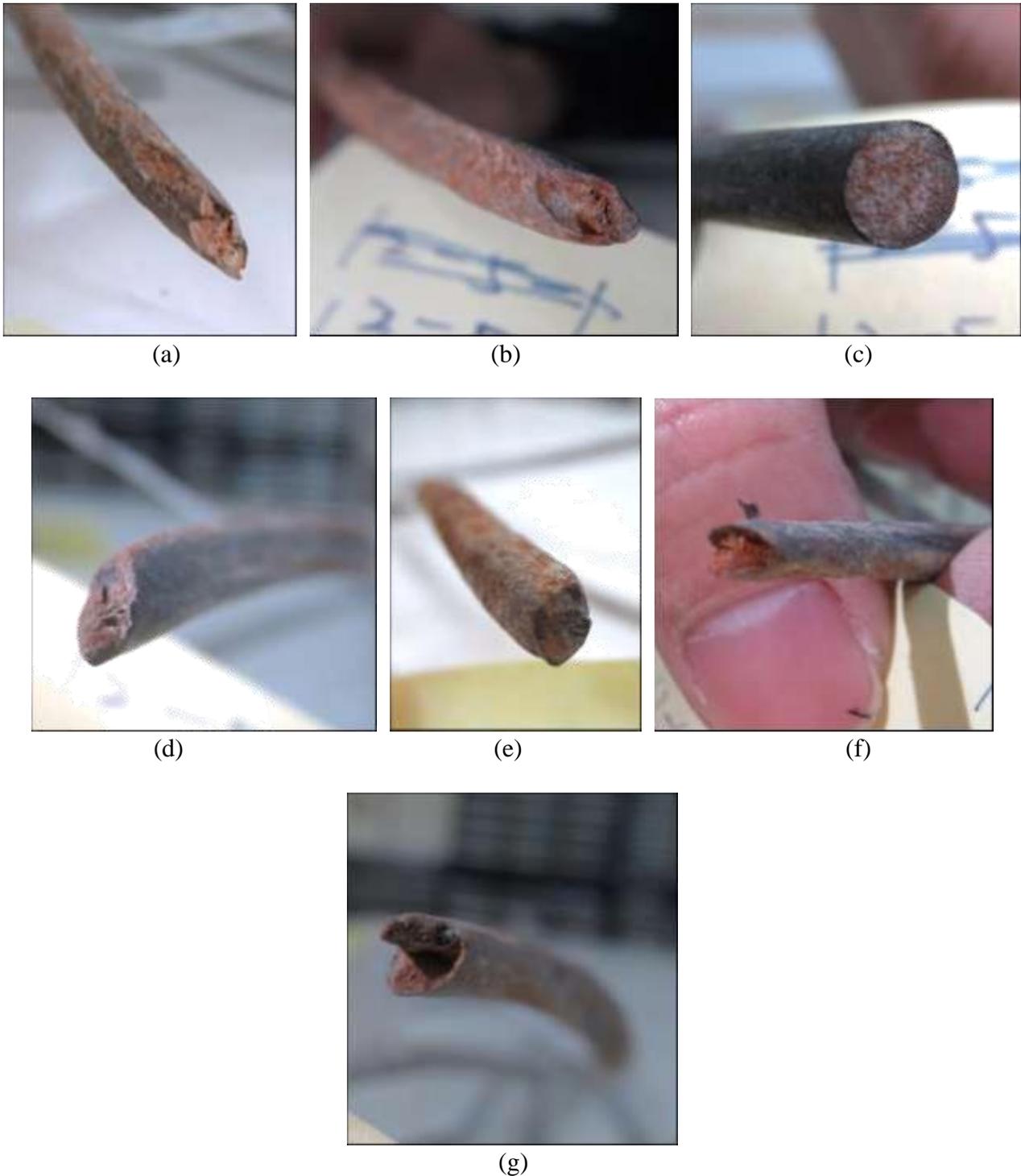


圖40 編號第 12 股吊索第 5 束鋼絞線之各鋼線斷面外觀

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(a)



(b)



(c)



(d)



(e)



(f)



(g)

圖41 編號第 12 股吊索第 6 束鋼絞線之各鋼線斷面外觀



(a)



(b)



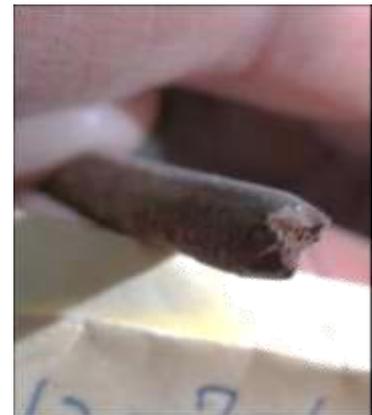
(c)



(d)



(e)



(f)



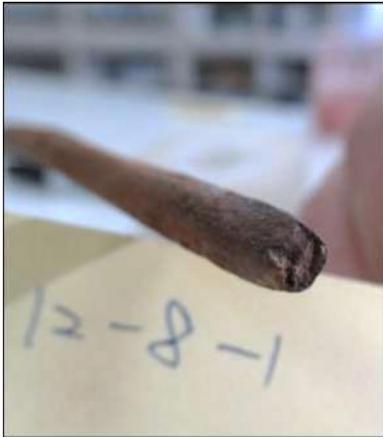
(g)

圖42 編號第 12 股吊索第 7 束鋼絞線之各鋼線斷面外觀

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(a)



(b)



(c)



(d)



(e)



(f)



(g)

圖43 編號第 12 股吊索第 8 束鋼絞線之各鋼線斷面外觀

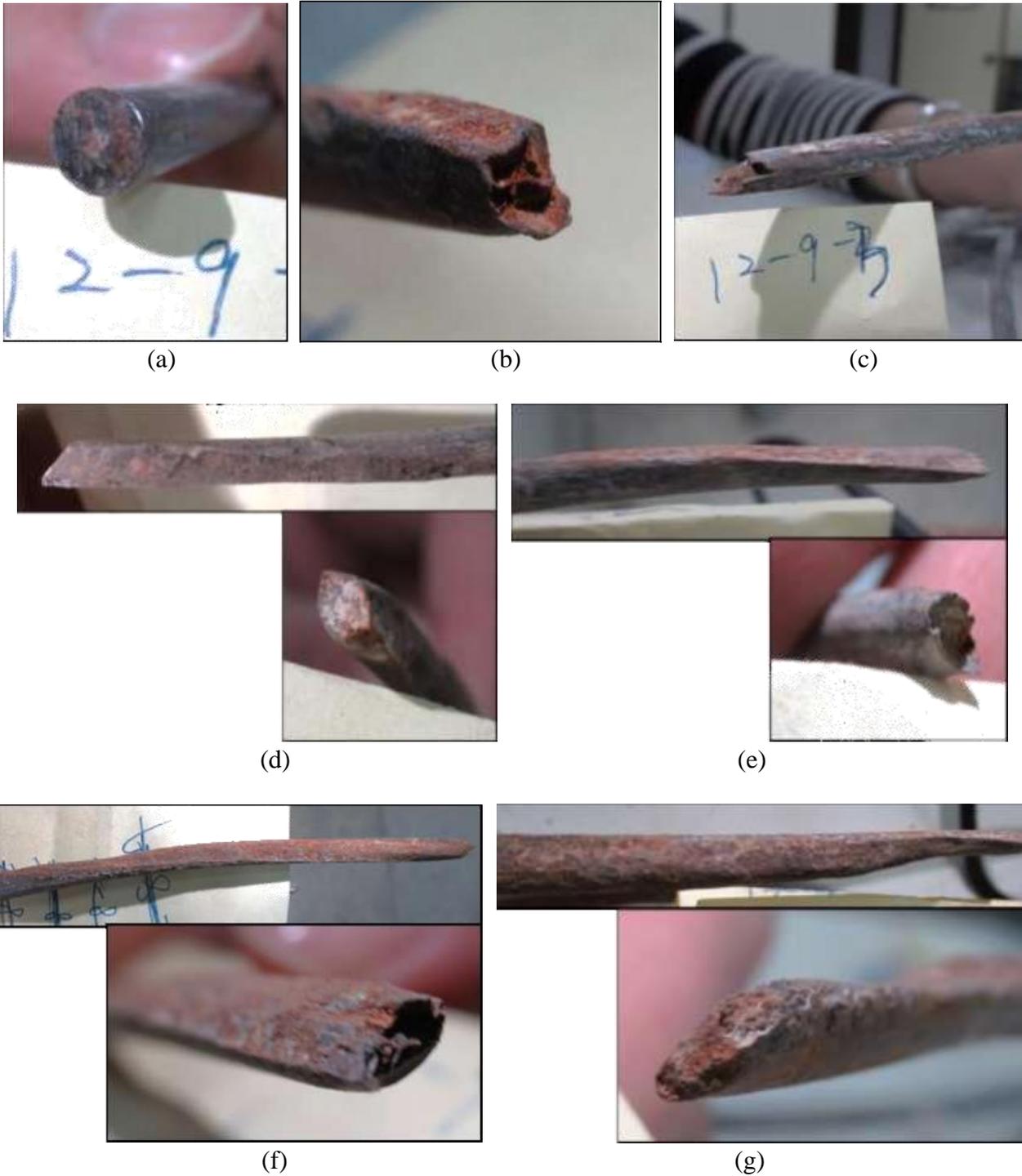


圖44 編號第 12 股吊索第 9 束鋼絞線之各鋼線斷面外觀

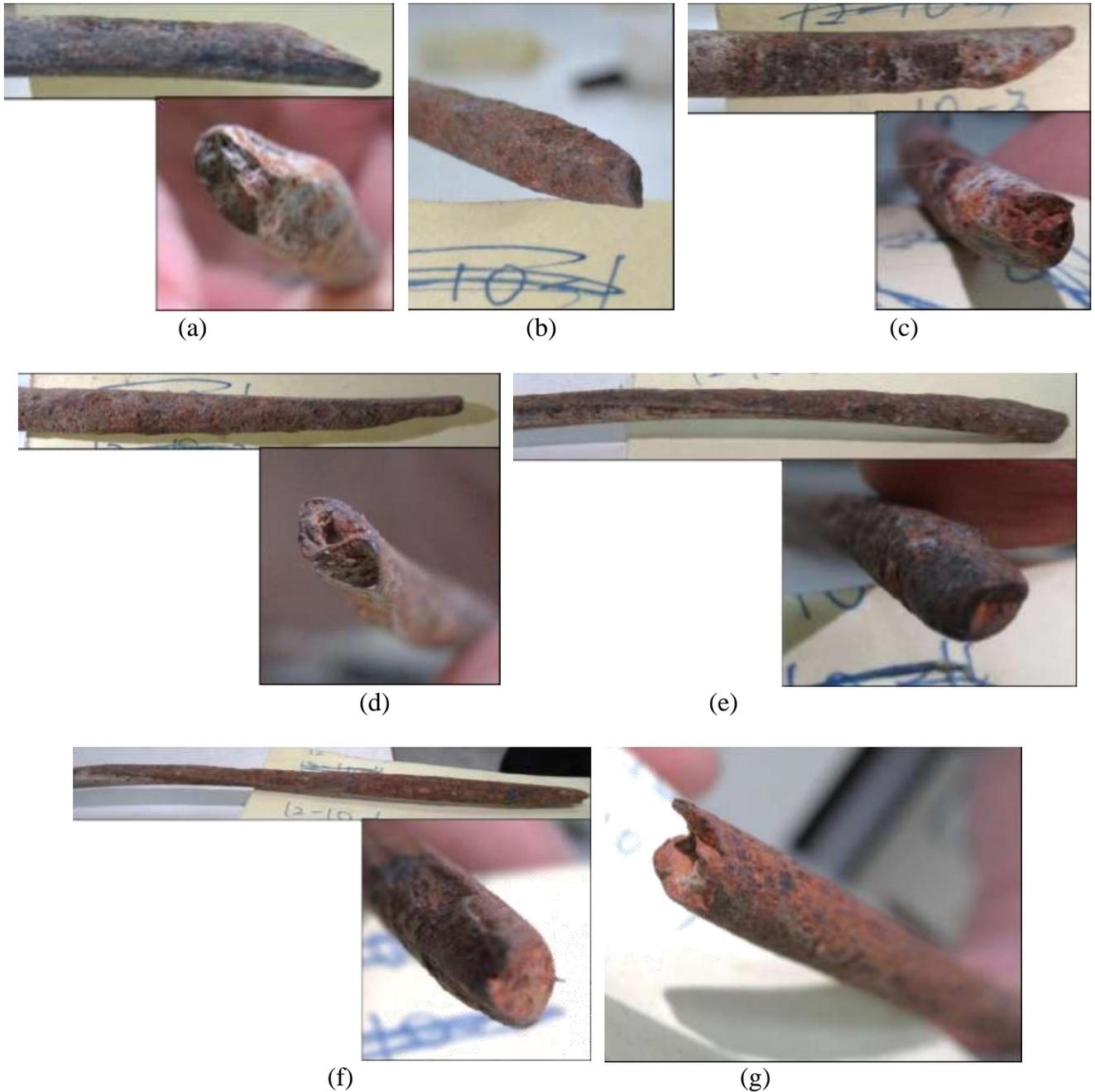


圖45 編號第 12 股吊索第 10 束鋼絞線之各鋼線斷面外觀

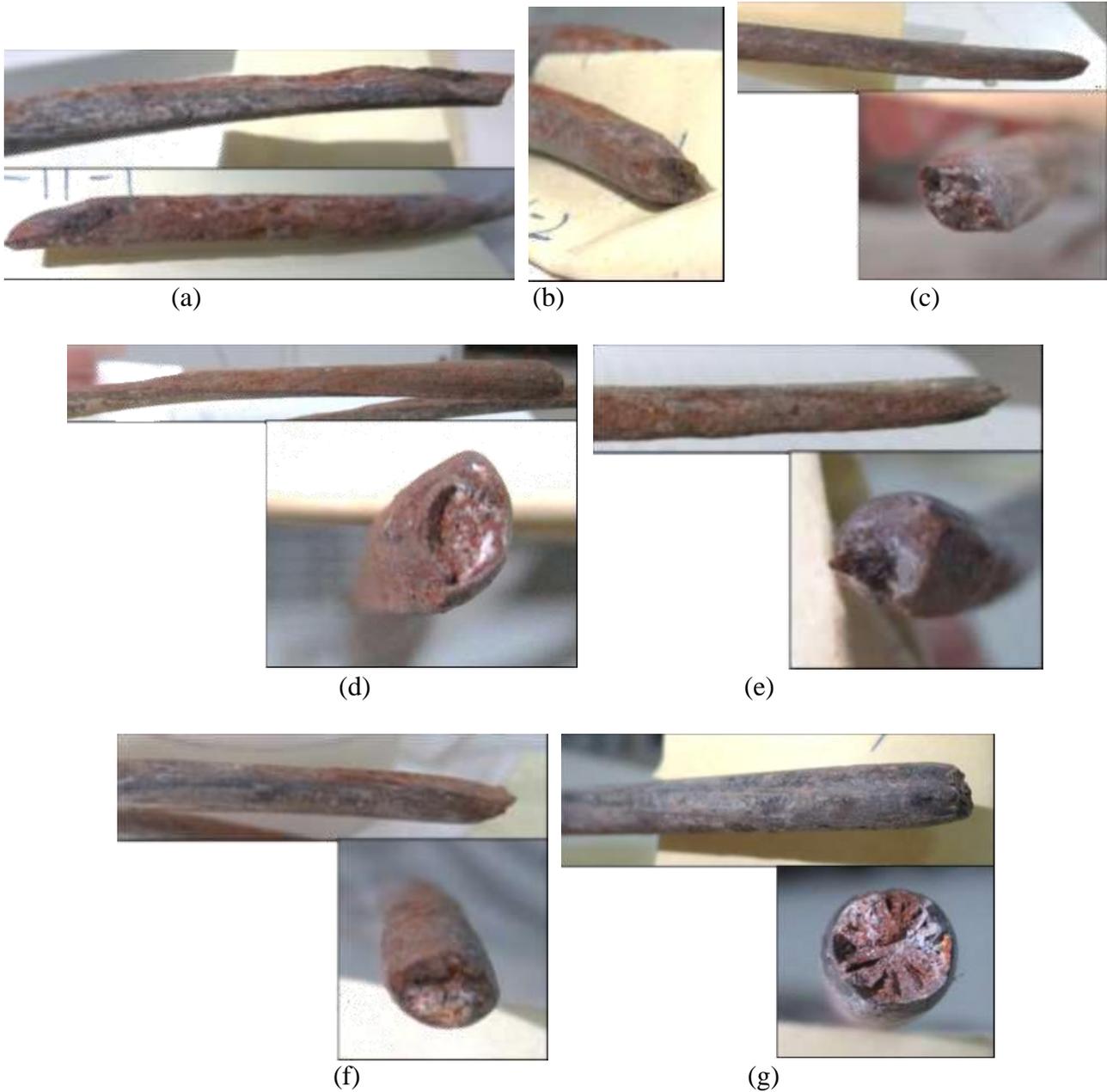


圖46 編號第 12 股吊索第 11 束鋼絞線之各鋼線斷面外觀

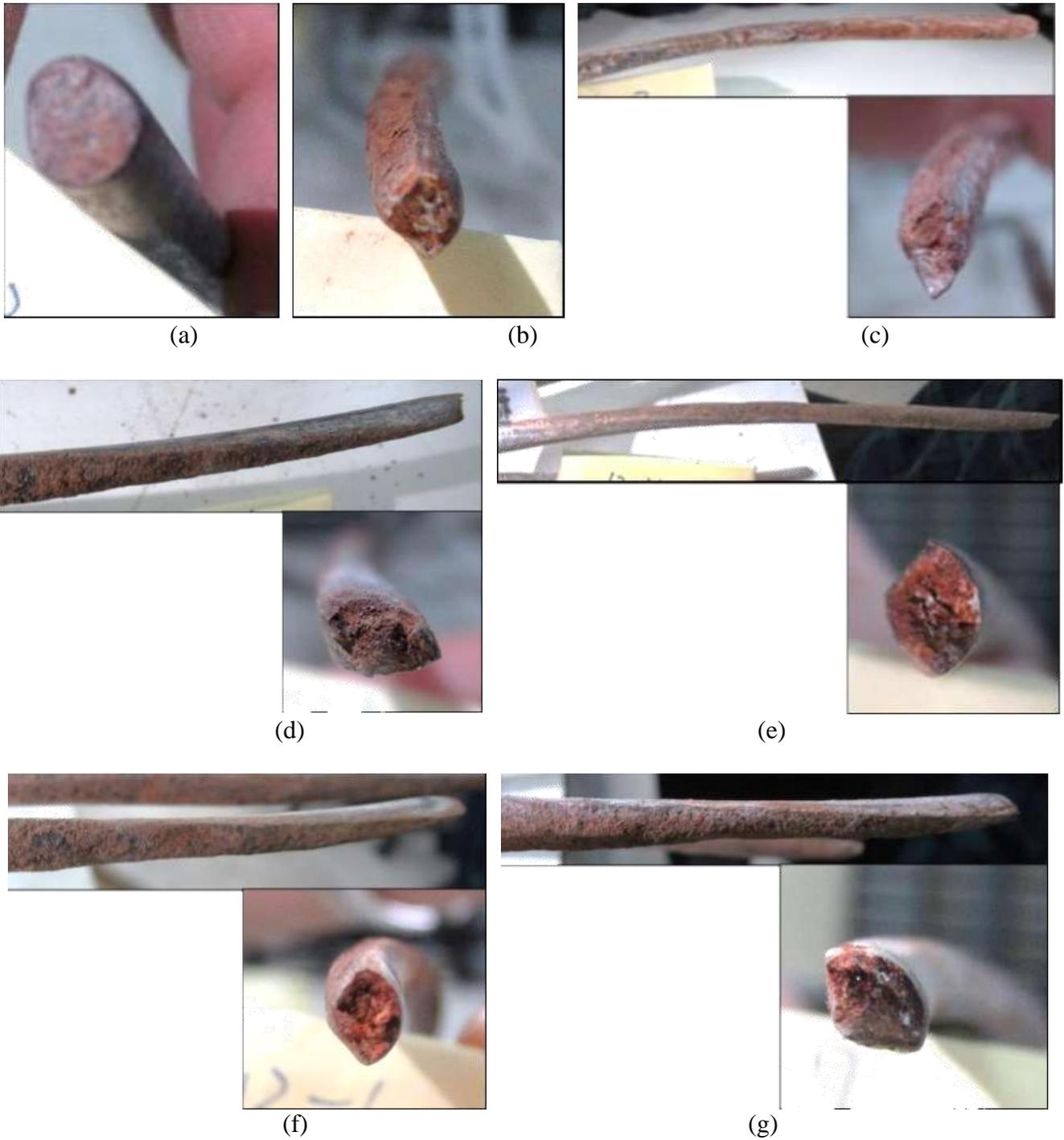


圖47 編號第 12 股吊索第 12 束鋼絞線之各鋼線斷面外觀



(a)



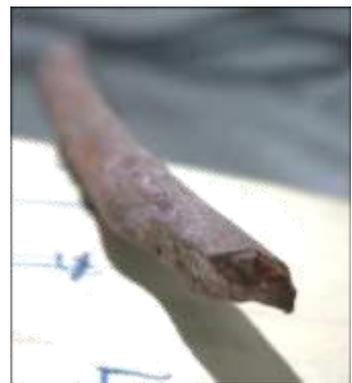
(b)



(c)



(d)



(e)



(f)



(g)

圖48 編號第 12 股吊索第 13 束鋼絞線之各鋼線斷面外觀

## 七、鋼線斷面 SEM 觀察

由於一般相機對於斷面細部觀察之解析度有限，而光學顯微鏡之景深較淺，亦無法對斷裂面進行細部觀察，故以掃描式電子顯微鏡(SEM)對各鋼線之斷面進行觀察。因鋼線試樣數量多，照片數量亦相當龐大，故本節僅列出具代表性之不同類型斷裂面照片，如圖 49～圖 54所示。圖 49為編號 8-2-1 鋼線之斷面 SEM 照片，為平整之斷面，並且可觀察到平行之加工痕跡，此種斷面多出現在芯線，平行加工痕跡為鋼線原始切割斷面，此類斷面顯示鋼線並非受拉伸斷裂，而是由鋼絞線束中抽出，故仍保留原始平整斷面。圖 50為編號 8-2-2 鋼線斷面 SEM 照片，該斷面為一斜面且較平整，為分解應力較大之平面斷裂，偏向脆性斷裂，但亦可觀察到其表面不平整，已有受表面鏽蝕影響。圖 51則為編號 9-4-2 鋼線斷面 SEM 照片，為不規則頸縮之形貌，顯示鋼線受拉伸應力時有發生頸縮，但因受鏽蝕影響而呈現不規則形貌。圖 52為編號 9-4-7 鋼線斷面 SEM 照片，為較對稱之頸縮斷面，但其斷面細部觀察可看到斷裂引起之裂隙。圖 53為編號 9-8-3 鋼線斷面 SEM 照片，屬於較複雜之斷面型態，疑似有頸縮變細之情形，但斷面又有偏向脆斷之形貌，研判亦有受鏽蝕之影響。圖 54為編號 9-2-5 鋼線斷面 SEM 照片，此種斷面可明顯看出為鏽蝕影響程度較大之斷裂面，斷面上並可觀察到較深之鏽蝕孔。

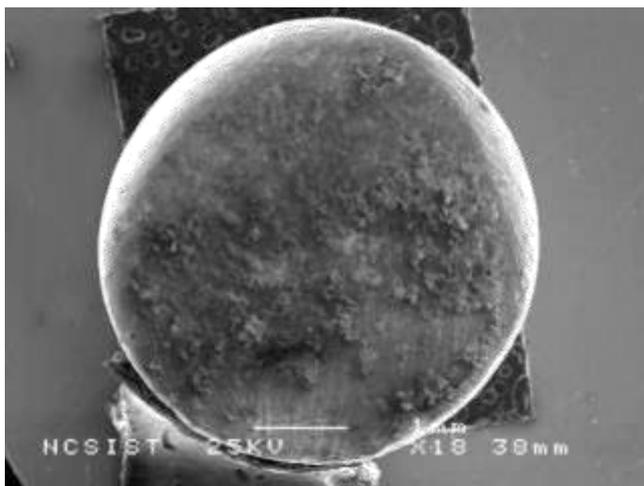


圖49 編號 8-2-1 鋼線(芯線)斷面 SEM 照片

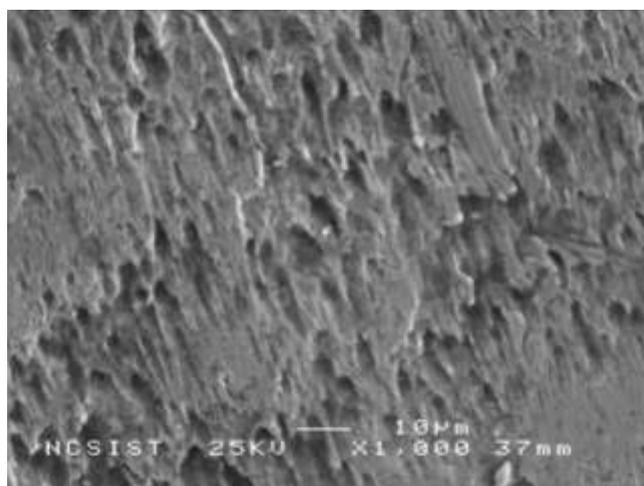
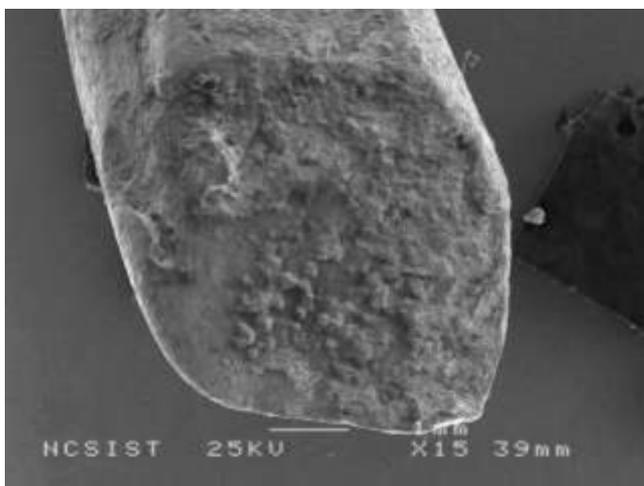
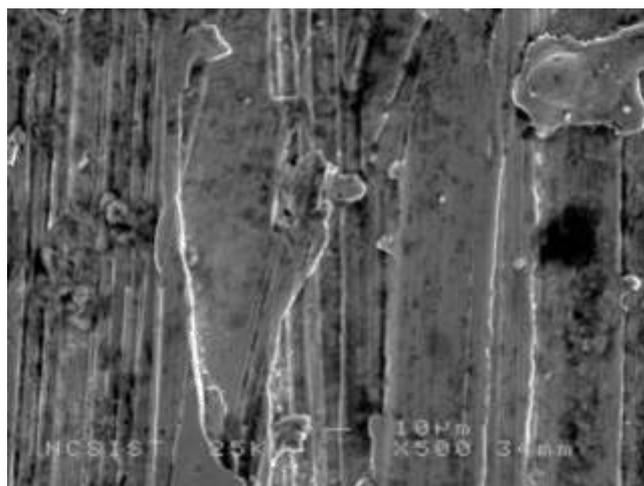


圖50 編號 8-2-2 鋼線斷面 SEM 照片

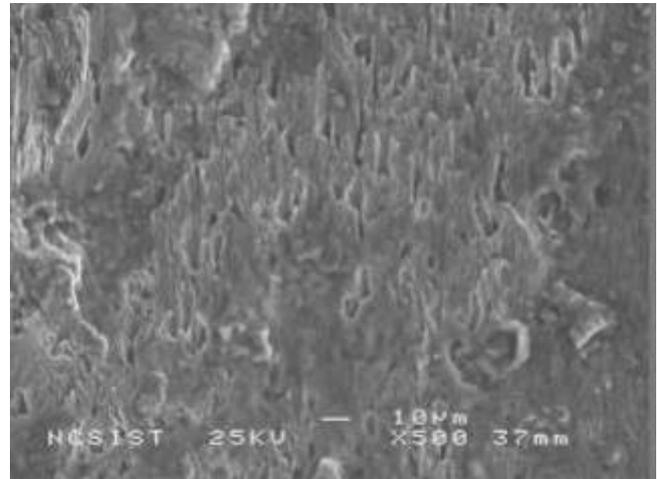
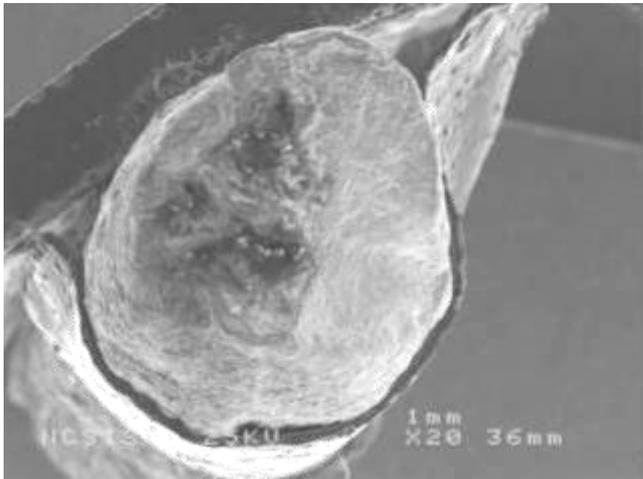


圖51 編號 9-4-2 鋼線斷面 SEM 照片

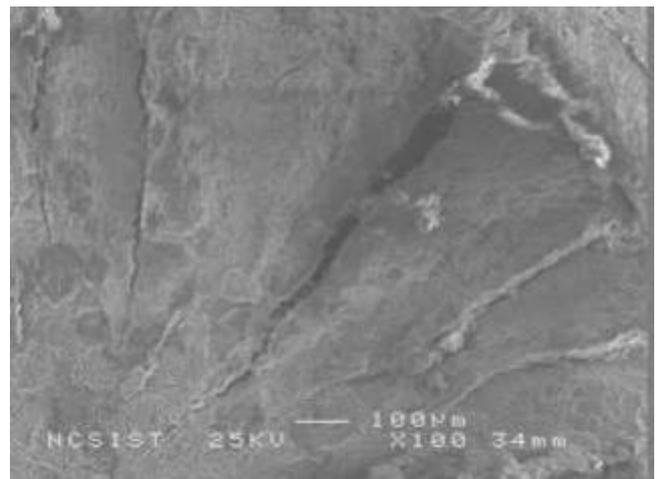
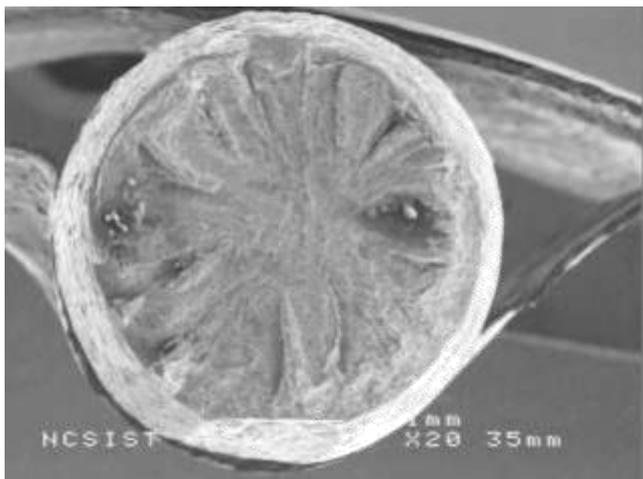


圖52 編號 9-4-7 鋼線斷面 SEM 照片

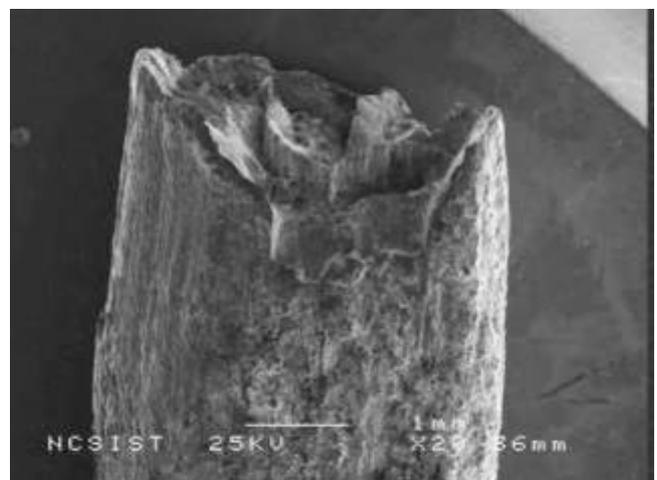


圖53 編號 9-8-3 鋼線斷面 SEM 照片

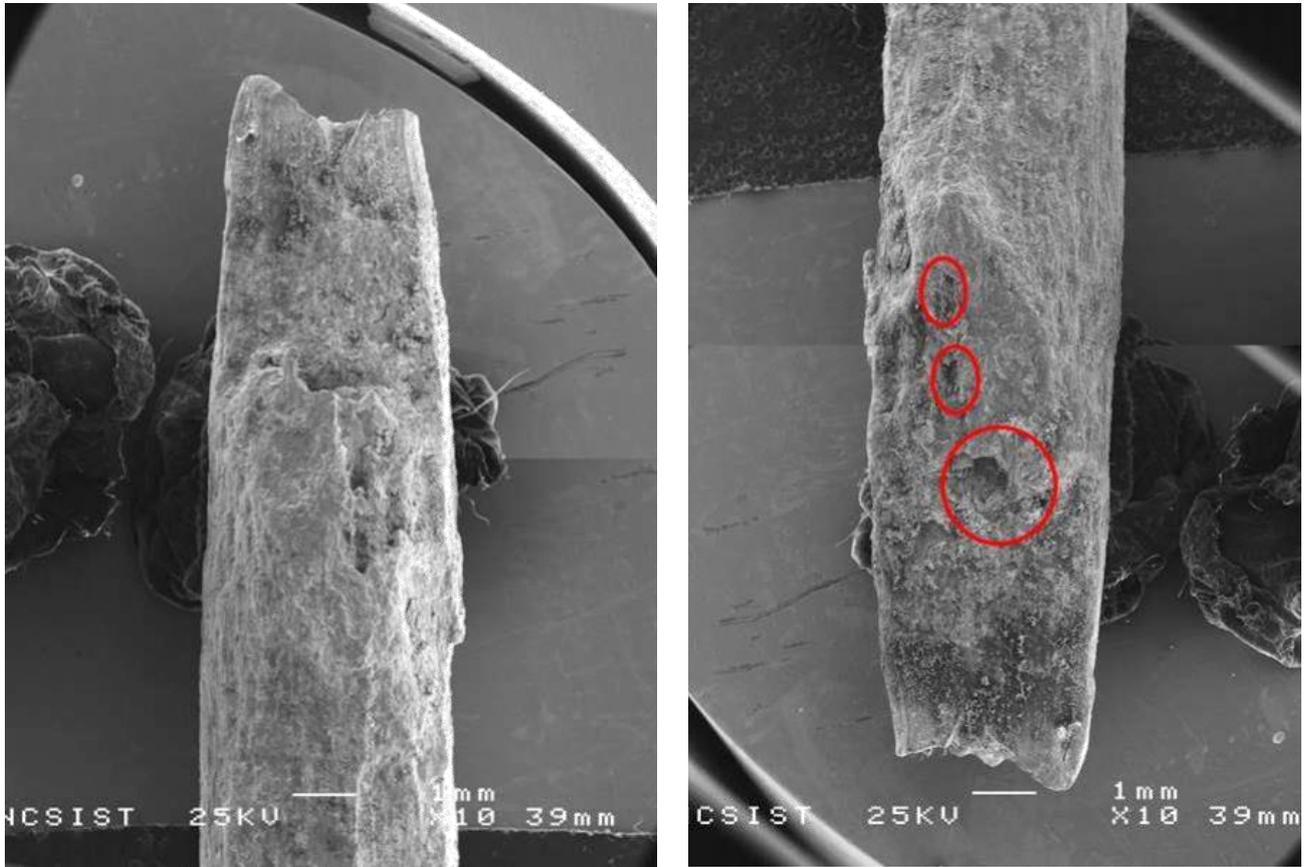


圖54 編號 9-2-5 鋼線斷面 SEM 照片

#### 八、鋼線成份分析

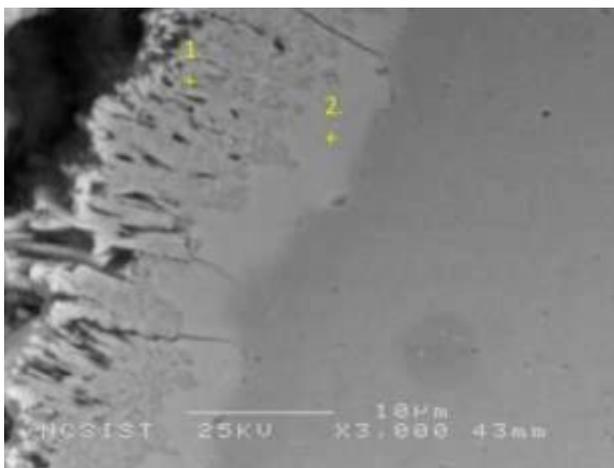
將編號 8、9、12 號吊索之鋼絞線分別取芯線及邊線進行成份分析，其中 9 號吊索之鋼絞線分別取全段生鏽及未生鏽之線束進行分析。將取樣後之鋼絞線以碳硫分析儀進行碳硫含量分析，另取剖面試樣經鑲埋、研磨及拋光後，以電子束顯微分析儀(EPMA)進行定量分析，所得結果彙整如表 3 所示。由各線材之成份差異，如碳含量、合金元素(如 Cr)含量差異，研判不同線束可能來自不同商源或不同批次之產品。

表3 鋼線試樣之成份分析結果

試樣編號	元素(wt%)									
	C	Si	Mn	P	S	Ni	Cr	Al	Cu	Fe
8-2-2(邊)	1.03	0.23	0.83	N/A	0.013	0.01	0.02	0.00	0.01	Rem.
8-2-1(芯)	1.07	0.19	0.69	N/A	0.010	0.00	0.25	0.00	0.06	Rem.
9-2-(邊)(生鏽)	0.81	0.22	0.87	N/A	0.004	0.00	0.20	0.02	0.00	Rem.
9-2-(芯)(生鏽)	0.82	0.22	0.86	N/A	0.008	0.00	0.20	0.01	0.01	Rem.
9-5-2(邊)(生鏽)	N/A	0.22	0.90	0.013	N/A	0.00	0.21	0.03	0.01	Rem.
9-5-5(邊)(生鏽)	N/A	0.21	0.85	0.011	N/A	0.00	0.22	0.03	0.00	Rem.
9-5-7(芯)(生鏽)	N/A	0.22	0.88	0.012	N/A	0.01	0.21	0.01	0.01	Rem.
9-8-(邊)(無生鏽)	0.86	0.21	0.73	N/A	0.013	0.00	0.01	0.00	0.01	Rem.
9-8-(芯)(無生鏽)	0.82	0.22	0.84	N/A	0.017	0.00	0.23	0.00	0.01	Rem.
9-4-1(邊)(無生鏽)	N/A	0.21	0.69	0.023	N/A	0.00	0.02	0.00	0.00	Rem.
9-4-2(邊)(無生鏽)	N/A	0.20	0.71	0.024	N/A	0.00	0.02	0.01	0.00	Rem.
9-4-7(芯)(無生鏽)	N/A	0.21	0.84	0.010	N/A	0.01	0.25	0.00	0.02	Rem.
12-7-1(邊)	1.07	0.23	0.83	N/A	0.013	0.00	0.02	0.00	0.04	Rem.
12-7-7(芯)	1.02	0.19	0.69	N/A	0.010	0.01	0.15	0.00	0.03	Rem.
12-11-1(邊)	1.09	0.21	0.70	N/A	0.010	0.01	0.02	0.00	0.04	Rem.
12-11-7(芯)	1.02	0.20	0.84	N/A	0.012	0.00	0.16	0.00	0.00	Rem.
AISI 1080 規格	0.75~ 0.88	--	0.60~ 0.90	0.040 Max.	0.050 Max.	--	0.20 Max.	--	--	Rem.

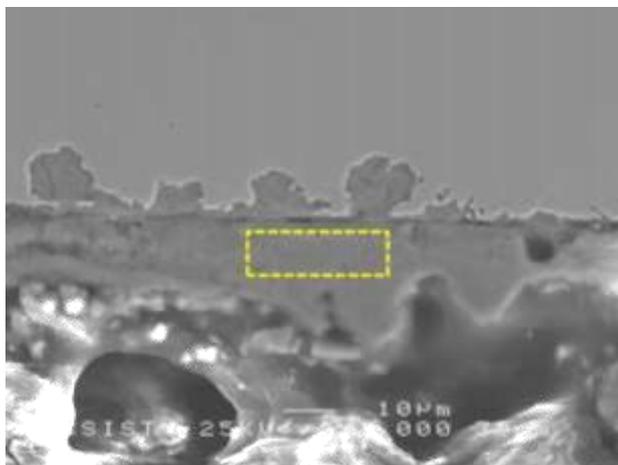
### 九、鋼線表面鍍層分析

為釐清不同線束表面鍍層之狀況，且由外觀檢視可發現各鋼絞線束之芯線表面氧化程度較少，故取不同線束之芯線進行表面鍍層分析，其中編號 9 號吊索亦取全段生鏽及未生鏽之線束。取樣後針對線材橫剖面進行鑲埋、研磨及拋光後，以掃描式電子顯微鏡(SEM) 搭配能量散佈光譜儀(EDS)進行元素含量分析，以了解表面鍍層狀況。圖 55 為 8 號吊索鋼絞線芯線之分析結果，可明顯觀察到表面具有鍍鋅層，但鋅層厚度僅剩約 10 μm，且因氧化嚴重導致鋅層結構疏鬆，僅鋅層與鋼材界面處較密實。圖 56 為 9 號吊索全段生鏽之鋼絞線芯線分析結果，可知表面僅有氧化層，並未發現鍍層，且因氧化嚴重，導致氧化層以侵入鋼材導致其表面不平整。圖 57 為 9 號吊索全段未生鏽之鋼絞線芯線分析結果，可發現其表面具有鍍鋅層，惟其形態亦不是很密實，可能受長時間氧化影響，鋅層厚度約接近 10 μm，但各處厚度不是很均勻。圖 58 為 12 號吊索鋼絞線芯線之分析結果，可發現亦有鍍鋅層存在，且含鋅層厚度可達約 30~40 μm，但可明顯看出有分層現象，較外層氧含量較高，顯示亦受長時間氧化影響。



1.	Element	Weight%	2.	Element	Weight%
	Zn	93.51		Zn	90.99
	Fe	6.49		Fe	9.01
Totals		100.00	Totals		100.00

圖55 編號 8-2 鋼線(芯線)剖面鍍層區 SEM 及 EDS 半定量分析結果



Element	Weight%
Fe	58.85
O	36.61
C	3.80
Mn	0.40
Al	0.34
Totals	100.00

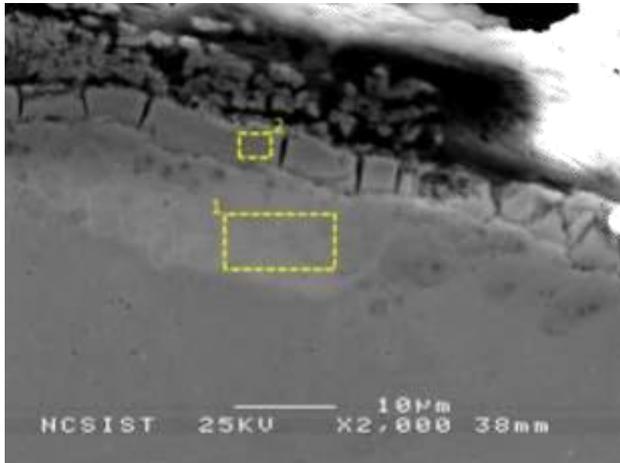
圖56 編號 9-2 生鏽鋼線(芯線)剖面鍍層區 SEM 及 EDS 半定量分析結果

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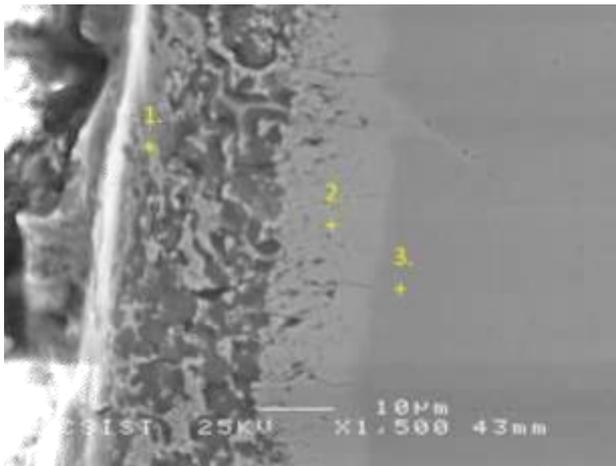
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1.		2.	
Element	Weight%	Element	Weight%
Fe	95.00	Zn	86.20
C	2.71	Fe	10.45
Zn	1.07	O	3.35
Mn	0.84		
Cr	0.37	Totals	100.00
Totals	100.00		

圖57 編號 9-8 未生鏽鋼線(芯線)剖面鍍層區 SEM 及 EDS 半定量分析結果



1.		2.	
Element	Weight%	Element	Weight%
Zn	75.28	Zn	92.77
O	22.54	Fe	7.23
Fe	1.06	Totals	100.00
S	0.57		
Ca	0.55		
Totals	100.00		

3.	
Element	Weight%
Fe	98.13
Zn	1.05
Mn	0.83
Totals	100.00

圖58 編號 12-7 未生鏽鋼線(芯線)剖面鍍層區 SEM 及 EDS 半定量分析結果

### 十、鋼線拉伸試驗

為了解鋼線之強度，取不同鋼絞線之鋼線(包含芯線及邊線)進行拉伸試驗，由於線材本身並非平直線材，伸長率並無法準確量測，且曾經使用受力變形，所得降伏強度可能已非原值，故拉伸測試主要以抗拉強度作為不同線材間之比較依據。拉伸試驗結果彙整如表 4 所示，可知不同線材間之抗拉強度差異不大，但可發現編號 9 號吊索之全段生鏽鋼絞線之線材抗拉強度普遍較低，僅芯線較高，顯示可能有受表面生鏽影響導致強度降低。

表4 鋼線試樣之拉伸試驗結果

試件編號	線徑(mm)	降伏強度(MPa)	抗拉強度(MPa)	伸長率(%)	斷裂位置
8-2-1(芯)	5.40	1373	1814	13.7	正常
8-2-2	5.18	1493	1906	15.0	正常
8-2-5	5.20	1394	1830	9.8	夾持端附近
9-4-1	5.20	1453	1910	12.5	正常
9-4-2	5.18	1428	1883	9.6	夾持端附近
9-4-7(芯)	5.30	1467	1857	5.6	夾持端附近
9-8-2	5.19	1463	1862	10.1	夾持端附近
9-8-3	5.19	1430	1905	12.5	正常
9-8-7(芯)	5.38	1422	1821	10.7	正常
9-2-1(鏽)	5.08	1343	1760	8.7	夾持端附近
9-2-5(鏽)	5.08	1424	1574	4.3	正常
9-2-7(芯)(鏽)	5.16	1336	1892	6.4	夾持端附近
9-5-2(鏽)	5.08	1389	1840	10.8	正常
9-5-5(鏽)	5.08	1358	1764	7.1	夾持端附近
9-5-7(芯)(鏽)	5.23	1432	1901	9.5	正常
12-7-1	5.18	1430	1942	12.7	正常
12-7-5	5.18	1422	1870	9.3	夾持端附近
12-7-7(芯)	5.37	1458	1837	10.0	夾持端附近
12-11-1	5.20	1497	1879	7.7	夾持端附近
12-11-2	5.20	1497	1901	5.7	夾持端附近
12-11-7(芯)	5.38	1425	1827	8.7	夾持端附近

### 十一、鋼線硬度測試

將不同鋼絞線取芯線及邊線進行剖面鑲埋、研磨及拋光後，以維氏硬度機進行硬度測試，荷重採用 300 gf，每鋼線隨機測試五點，所得結果如表 5 所示。由測試結果可知不同鋼絞線之線材硬度差異不大，惟編號 12 號吊索之鋼絞線硬度有略低之現象。

表5 鋼線試樣之硬度測試結果(HV0.3)

試片編號	點 1	點 2	點 3	點 4	點 5	平均值
8-2-1(芯)	481	468	492	495	495	486
8-2-2(邊)	494	491	501	490	483	492
9-8(邊)	508	504	506	511	508	507
9-8(芯)	508	503	503	508	508	506
9-4(邊)	511	503	505	492	508	504
9-4(芯)	472	479	489	492	500	486
9-2(邊)(鏽)	480	491	494	492	482	488
9-2(芯)(鏽)	496	503	507	501	501	502
9-5(邊)(鏽)	498	511	509	491	511	504
9-5(芯)(鏽)	496	493	479	504	509	496
12-7-1(邊)	512	500	491	503	512	504
12-7-7(芯)	486	495	492	473	470	483
12-11-1(邊)	483	463	467	470	491	475
12-11-7(芯)	461	469	462	459	476	465

### 十二、橋面端試樣

橋面端試樣因事故發生時部份隨橋體一同墜入海中，故部份橋面端試樣浸泡在海水中一段時間後才被打撈起，橋面 1~13 號錨頭外觀如圖 59~圖 71 所示。由於許多橋面端錨頭仍有鋼絞線殘留其上，且多數仍與承壓環及螺紋調整管相連，故經外觀檢視後，即送至國震中心進行拆解，將各部件分離，再送至本院進行相關分析。圖 72~圖 84 為拆解後橋面 1~13 號錨頭外觀照片，可知除橋面 1 號錨頭破裂外，其餘橋面 2~13 號錨頭並無破裂，但皆有生鏽情況，惟各錨頭生鏽程度略有差異。此外，隨同拆解後錨頭一併送至本院的部件尚有各錨頭拆解後之相關配件，包含承壓板、殘留之鋼絞線、夾片等，各編號橋面錨頭之相關配件外觀照片則如圖 85~圖 97 所示。



圖59 橋面 1 號錨頭拆解前外觀



圖60 橋面 2 號錨頭拆解前外觀



圖61 橋面 3 號錨頭拆解前外觀



圖62 橋面 4 號錨頭拆解前外觀



圖63 橋面 5 號錨頭拆解前外觀

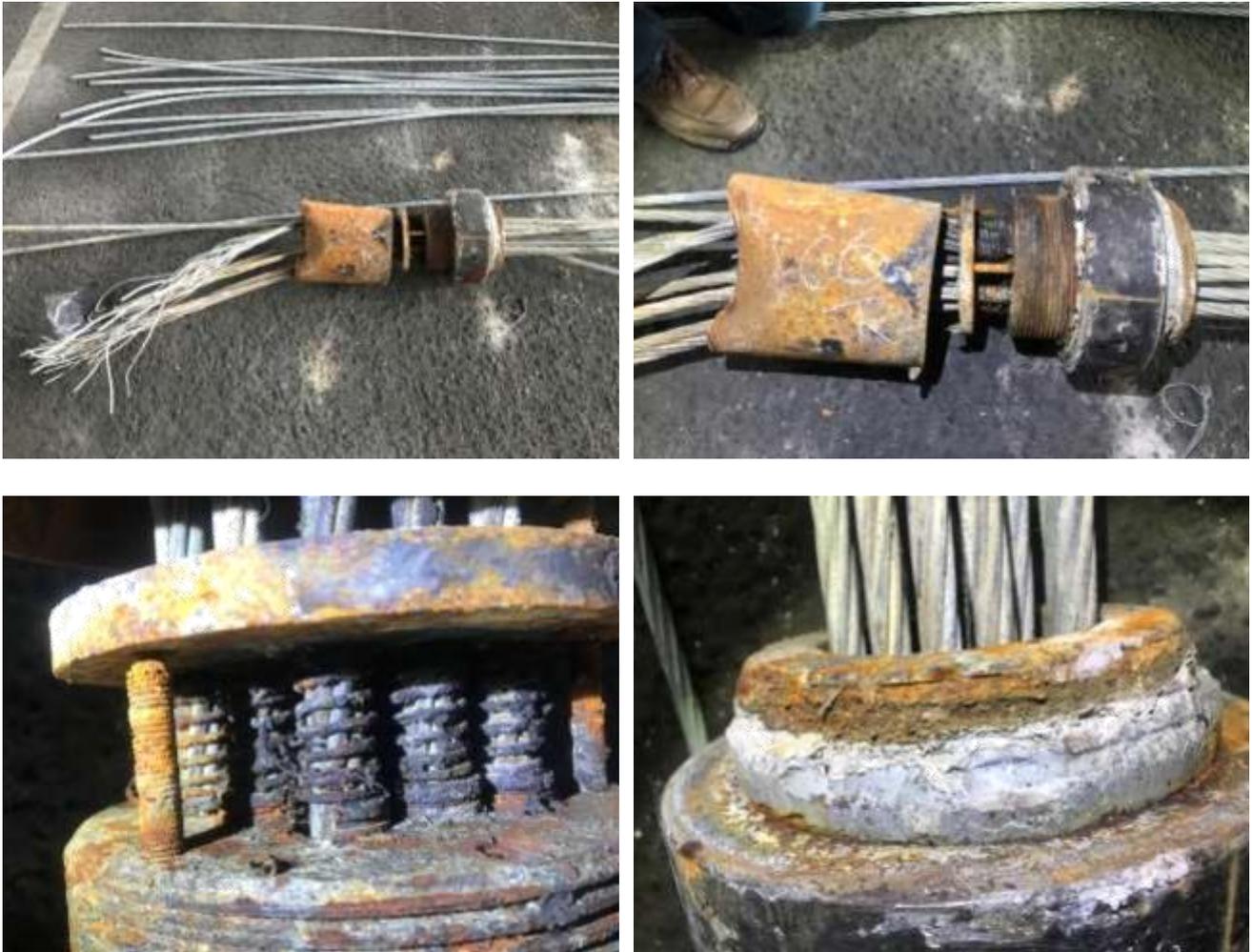


圖64 橋面 6 號錨頭拆解前外觀



圖65 橋面 7 號錨頭拆解前外觀



圖66 橋面 8 號錨頭拆解前外觀



圖67 橋面 9 號錨頭拆解前外觀



圖68 橋面 10 號錨頭拆解前外觀



圖69 橋面 11 號錨頭拆解前外觀



圖70 橋面 12 號錨頭拆解前外觀



圖71 橋面 13 號錨頭拆解前外觀



圖72 橋面 1 號錨頭拆解後外觀



圖73 橋面 2 號錨頭拆解後外觀



圖74 橋面 3 號錨頭拆解後外觀



圖75 橋面 4 號錨頭拆解後外觀



圖76 橋面 5 號錨頭拆解後外觀



圖77 橋面 6 號錨頭拆解後外觀

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圖78 橋面 7 號錨頭拆解後外觀



圖79 橋面 8 號錨頭拆解後外觀



圖80 橋面 9 號錨頭拆解後外觀



圖81 橋面 10 號錨頭拆解後外觀



圖82 橋面 11 號錨頭拆解後外觀



圖83 橋面 12 號錨頭拆解後外觀



圖84 橋面 13 號錨頭拆解後外觀



圖85 橋面 1 號錨頭拆解後其相關配件外觀



圖86 橋面 2 號錨頭拆解後其相關配件外觀



圖87 橋面 3 號錨頭拆解後其相關配件外觀



圖88 橋面 4 號錨頭拆解後其相關配件外觀

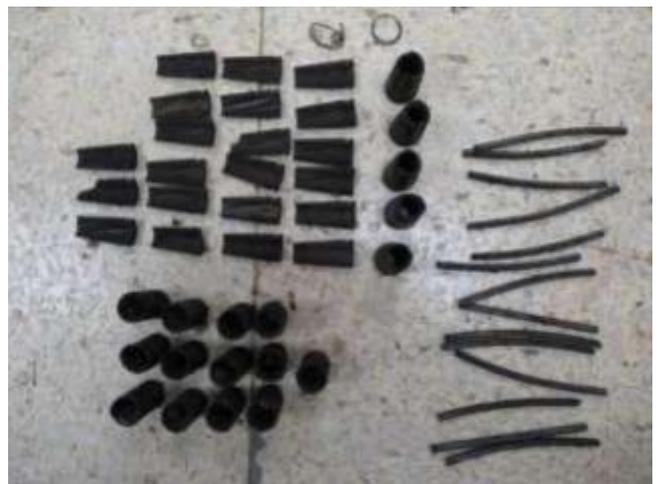


圖89 橋面 5 號錨頭拆解後其相關配件外觀

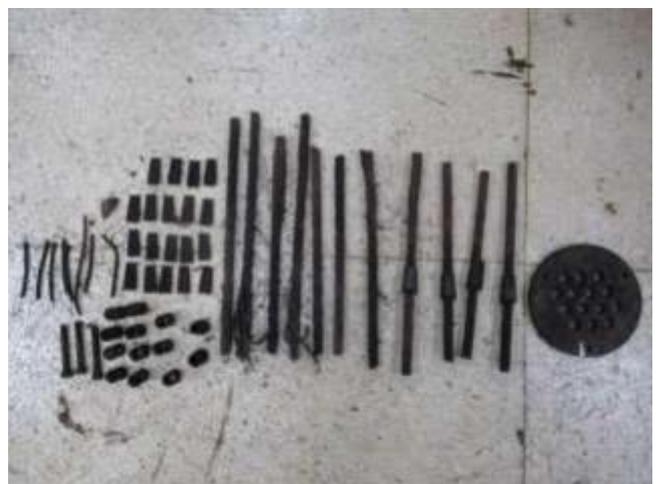




圖90 橋面 6 號錨頭拆解後其相關配件外觀



圖91 橋面 7 號錨頭拆解後其相關配件外觀



圖92 橋面 8 號錨頭拆解後其相關配件外觀





圖93 橋面 9 號錨頭拆解後其相關配件外觀



圖94 橋面 10 號錨頭拆解後其相關配件外觀



圖95 橋面 11 號錨頭拆解後其相關配件外觀



圖96 橋面 12 號錨頭拆解後其相關配件外觀



圖97 橋面 13 號錨頭拆解後其相關配件外觀

### 十三、 錨頭材質分析

取得橋面試樣後，為與上拱端錨頭進行比對，另選取特定編號之錨頭進行分析，各編號皆取上拱端及橋面端錨頭進行取樣以利比對；選取編號之考量項目包括孔數不同之 1 號(1 號及 13 號之孔數與其他編號錨頭不同)、上拱錨頭破裂之 3 號及 8 號、上拱未破裂之 9 號(上拱 7、9、11 號錨頭未破裂)及上拱未破裂且 13 束鋼絞線仍完整固定於其上之 10 號(上拱 10 號及 12 號之 13 束鋼絞線皆完整固定於其上)。將選取編號之上拱端及橋面端錨頭進行切割取樣，並進行材質分析、拉伸試驗及硬度測試。

將取樣後之錨頭試樣經研磨後，以火花發射光譜儀(SPARK-OES)進行成份分析，所得結果如表 6 所示，可發現不同編號之錨頭不論上拱端或橋面端，皆為相同材質，符合 JIS S45C 之規格。

表6 選定錨頭試樣之成份分析結果

試樣編號	元素(wt%)									
	C	Si	S	P	Mn	Ni	Cr	Mo	Cu	Fe
上拱 1 號	0.44	0.27	0.005	0.018	0.68	0.06	0.12	0.01	0.14	Rem.
上拱 3 號	0.45	0.27	0.006	0.017	0.69	0.06	0.12	0.01	0.15	Rem.
上拱 8 號	0.44	0.27	0.006	0.018	0.68	0.06	0.11	0.01	0.15	Rem.
上拱 9 號	0.44	0.27	0.005	0.017	0.68	0.06	0.12	0.01	0.14	Rem.
上拱 10 號	0.46	0.27	0.004	0.017	0.69	0.06	0.12	0.01	0.15	Rem.
橋面 1 號	0.45	0.27	0.002	0.017	0.68	0.05	0.11	0.01	0.14	Rem.
橋面 3 號	0.44	0.27	0.006	0.017	0.68	0.06	0.12	0.02	0.15	Rem.
橋面 8 號	0.44	0.27	0.007	0.019	0.68	0.06	0.12	0.02	0.15	Rem.
橋面 9 號	0.44	0.27	0.005	0.017	0.68	0.06	0.11	0.01	0.14	Rem.
橋面 10 號	0.44	0.27	0.005	0.019	0.68	0.06	0.12	0.02	0.14	Rem.
JIS G4051 S45C 規格	0.42~ 0.48	0.15~ 0.35	0.035 Max.	0.030 Max.	0.60~ 0.90	--	0.20 Max.	--	--	Rem.
SAE-AISI 1045 規格	0.43~ 0.50		0.050 Max.	0.040 Max.	0.60~ 0.90					Rem.

#### 十四、 錨頭材質拉伸試驗

將選取編號之上拱端及橋面端錨頭切割取樣，並依照 ASTM E8/E8M 規範加工為拉伸試棒進行拉伸試驗，每個錨頭測試三件，所得結果如表 7 及表 8 所示，可知各錨頭材質之強度相仿，偶有降伏強度較高之試桿，整體降伏強度平均值約 338 MPa，極限抗拉強度分佈較平均，整體抗拉強度平均值約 674 MPa，伸長率平均值約 22.1%。

表7 選定上拱錨頭試樣之拉伸試驗結果

試樣編號	降伏強度(MPa)	極限抗拉強度(MPa)	伸長率(%)
上拱 1 號#1	390	675	21.4
上拱 1 號#2	361	690	21.1
上拱 1 號#3	326	680	22.6
上拱 3 號#1	369	685	22.0
上拱 3 號#2	377	680	21.4
上拱 3 號#3	342	675	21.3
上拱 8 號#1	345	665	22.2
上拱 8 號#2	312	670	22.7
上拱 8 號#3	328	670	21.2
上拱 9 號#1	338	670	23.0
上拱 9 號#2	333	665	23.0
上拱 9 號#3	340	670	23.0
上拱 10 號#1	314	680	22.8
上拱 10 號#2	334	680	22.6
上拱 10 號#3	373	680	22.0
平均	345	676	22.2

表8 選定橋面錨頭試樣之拉伸試驗結果

試樣編號	降伏強度(MPa)	極限抗拉強度(MPa)	伸長率(%)
橋面 1 號#1	302	670	21.9
橋面 1 號#2	311	675	21.4
橋面 1 號#3	358	690	21.0
橋面 3 號#1	328	660	22.8
橋面 3 號#2	334	670	22.9
橋面 3 號#3	322	670	22.6
橋面 8 號#1	342	680	22.9
橋面 8 號#2	324	675	21.9
橋面 8 號#3	332	665	23.6
橋面 9 號#1	344	680	22.0
橋面 9 號#2	321	670	21.2
橋面 9 號#3	338	675	21.7
橋面 10 號#1	340	665	21.3
橋面 10 號#2	331	665	22.1
橋面 10 號#3	326	660	21.9
平均	330	671	22.1

### 十五、 錨頭材質硬度測試

將取樣後之錨頭試樣以洛氏硬度機進行硬度測試，每件試樣隨機測試五點，所得結果如表 9 所示，並列出換算 HB 之硬度值(參照 ASTM E140 規範)，可知各錨頭材質之硬度相近，部份錨頭硬度略高，整體硬度平均值約 91 HRBS。綜合拉伸試驗及硬度測試結果，研判本案錨頭使用之 S45C 材料並未經過淬回火熱處理達最高強度狀態。

表9 選定錨頭試樣之硬度測試結果

試樣編號	點 1	點 2	點 3	點 4	點 5	平均值	換算 HB
上拱 1 號	91.9	92.2	93.4	91.2	90.2	91.8	195
上拱 3 號	96.0	96.0	93.6	96.1	98.5	96.0	216
上拱 8 號	93.7	90.3	91.1	91.1	93.4	91.9	195
上拱 9 號	88.4	87.5	85.0	87.8	89.7	87.7	176
上拱 10 號	94.8	95.4	96.1	95.0	95.7	95.4	210
橋面 1 號	93.3	91.5	92.2	93.5	92.0	92.5	200
橋面 3 號	89.0	88.9	88.6	89.1	88.6	88.8	180
橋面 8 號	86.6	89.0	88.5	88.2	86.8	87.8	176
橋面 9 號	87.2	89.2	89.4	89.7	89.5	89.0	180
橋面 10 號	86.8	88.2	88.5	89.6	89.3	88.5	180

### 十六、 橋面端殘留鋼絞線外觀檢視

由前述圖 85~圖 97 可知送至本院之橋面端錨頭殘留之鋼絞線皆為較短之線段，主要因為殘留於橋面端錨頭之鋼絞線為較長之線段，經拆解後皆留存於國震中心以備鋼絞線強度測試應用。為了解鋼絞線之維護狀態與事故之相關性，亦針對橋面端錨頭殘留之鋼絞線進行分析，橋面端 7、9、10、11、12、13 號錨頭殘留之鋼絞線細部外觀檢查照片如所示。



圖98 橋面 7 號錨頭殘留之全部鋼絞線外觀



圖99 橋面 7 號錨頭殘留之鋼絞線 7-1 束外觀

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圖100 橋面 7 號錨頭殘留之鋼絞線 7-2 束外觀

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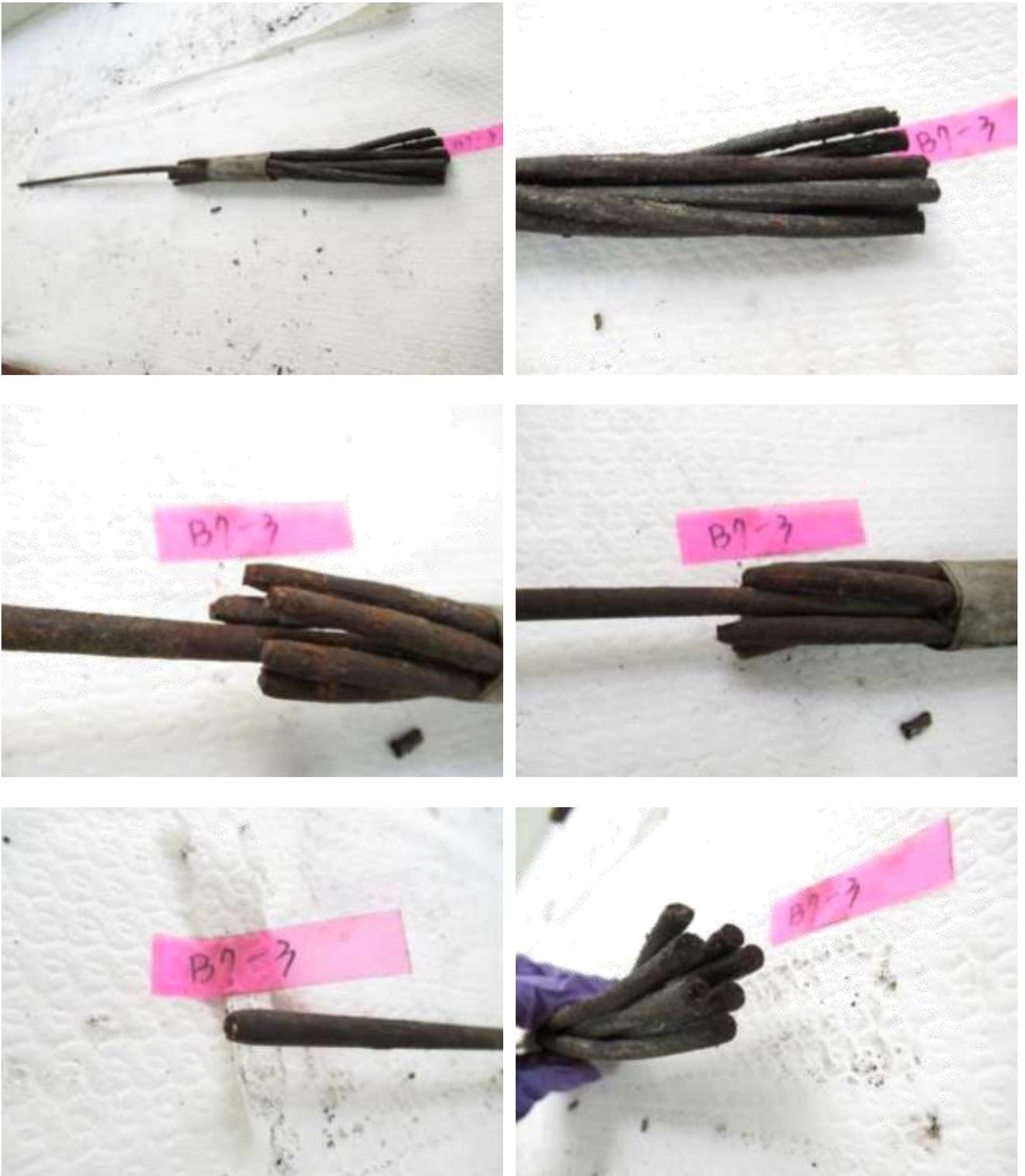


圖101 橋面 7 號錨頭殘留之鋼絞線 7-3 束外觀

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圖102 橋面 7 號錨頭殘留之鋼絞線 7-4 東外觀

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圖103 橋面 7 號錨頭殘留之散落鋼絞線外觀



圖104 橋面 9 號錨頭殘留之全部鋼絞線外觀

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圖105 橋面 9 號錨頭殘留之鋼絞線 9-1 束外觀

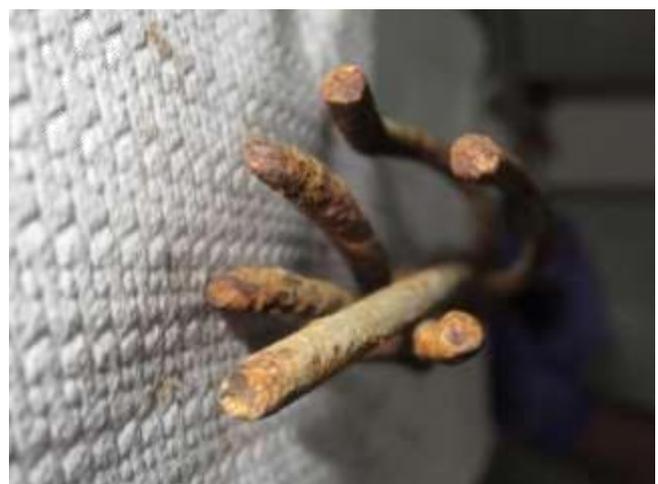
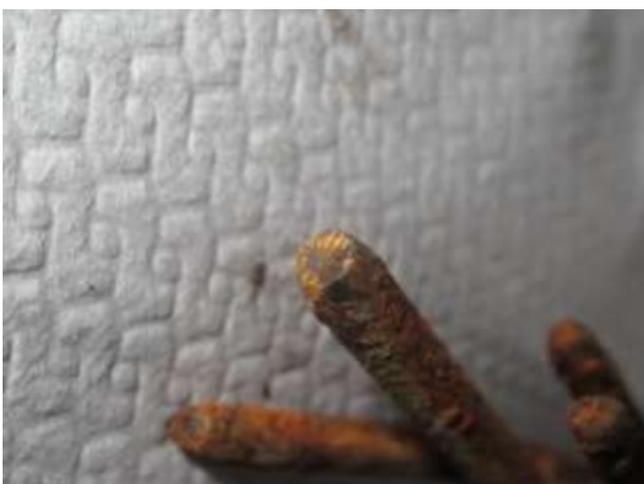
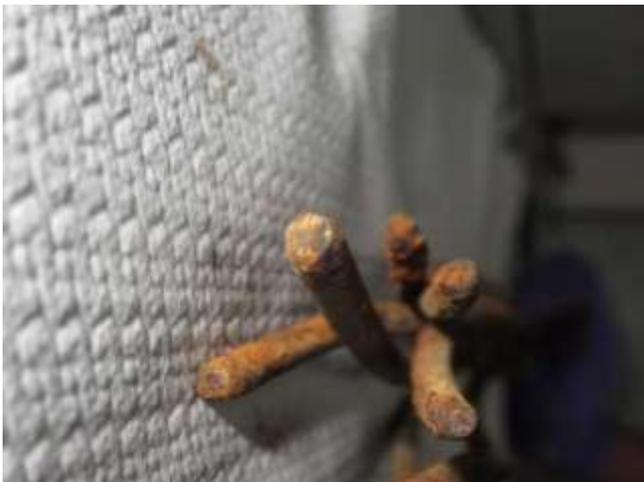


圖106 橋面 9 號錨頭殘留之鋼絞線 9-2 束外觀

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圖107 橋面9號錨頭殘留之鋼絞線9-3束外觀

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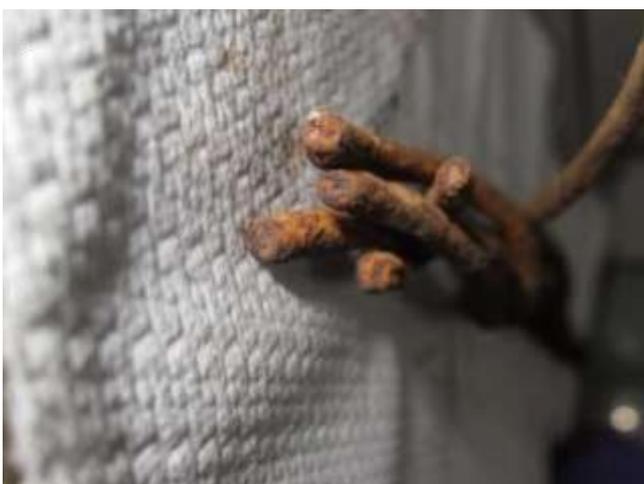


圖108 橋面 9 號錨頭殘留之鋼絞線 9-4 束外觀

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圖109 橋面 9 號錨頭殘留之鋼絞線 9-5 束外觀

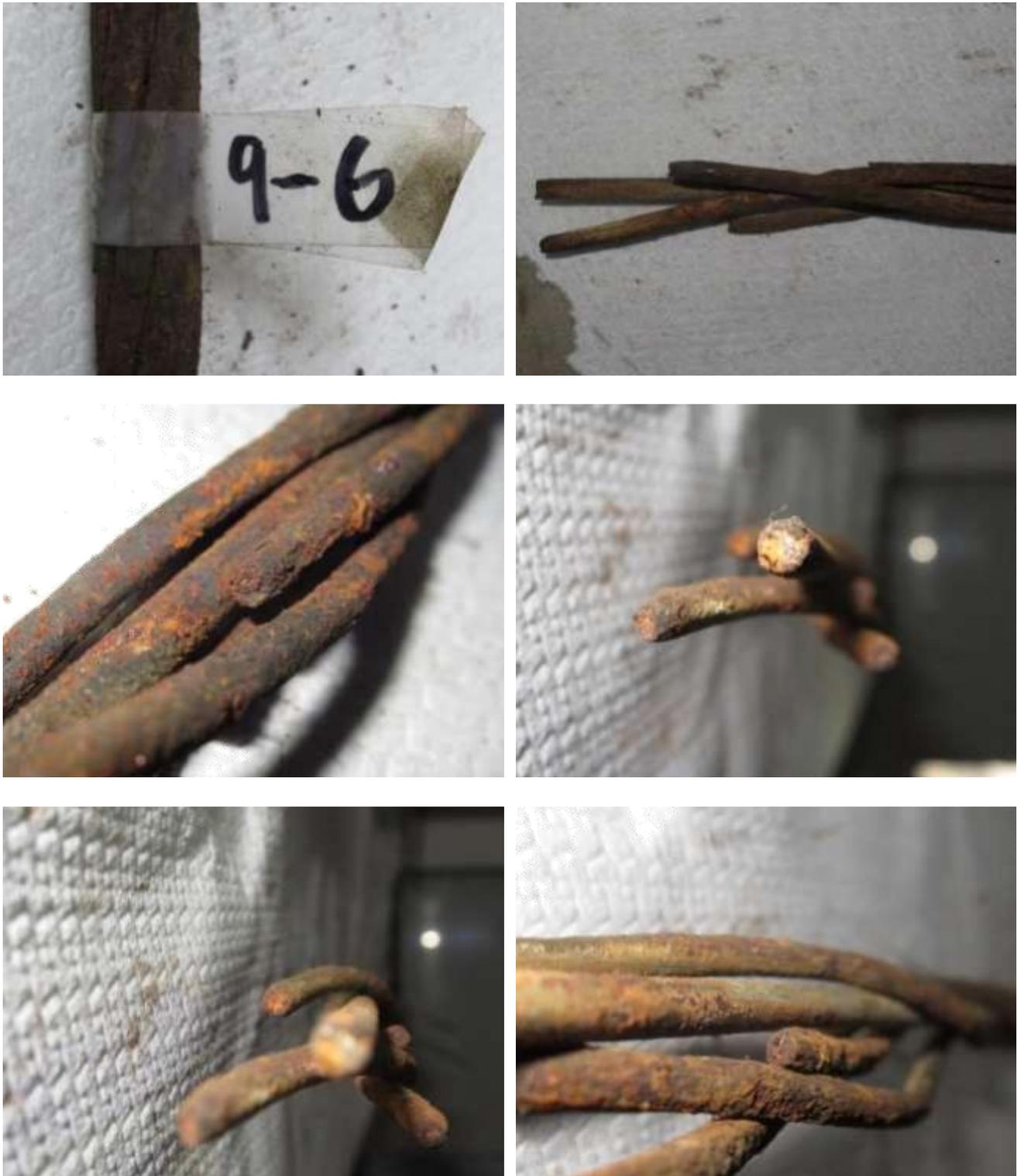


圖110 橋面 9 號錨頭殘留之鋼絞線 9-6 束外觀



圖111 橋面9號錨頭殘留之鋼絞線9-7束外觀



圖112 橋面 9 號錨頭殘留之鋼絞線 9-8 束外觀

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圖113 橋面 9 號錨頭殘留之鋼絞線 9-9 束外觀



圖114 橋面 9 號錨頭殘留之鋼絞線 9-10 束外觀



圖115 橋面 9 號錨頭殘留之鋼絞線 9-11 束外觀

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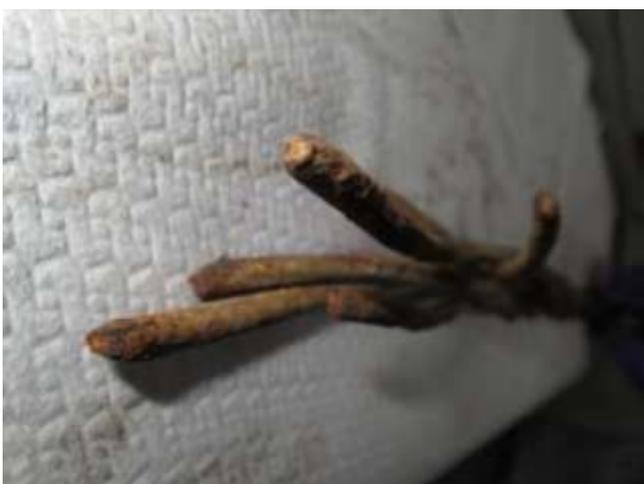
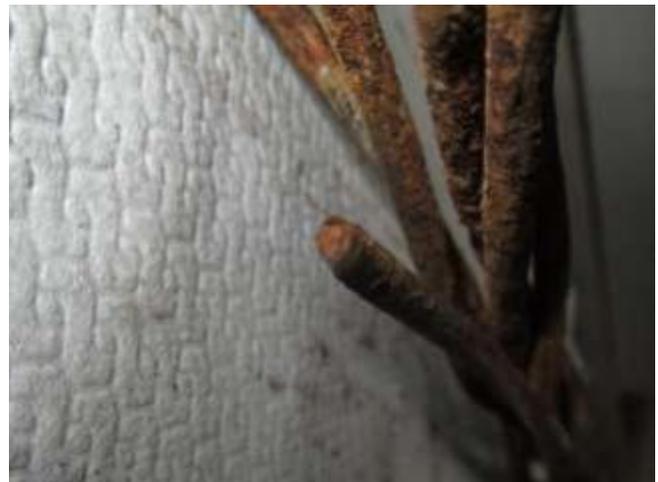


圖116 橋面 9 號錨頭殘留之鋼絞線 9-12 束外觀

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圖117 橋面 9 號錨頭殘留之鋼絞線 9-13 束外觀



圖118 橋面 10 號錨頭殘留之全部鋼絞線外觀

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圖119 橋面 10 號錨頭殘留之鋼絞線 10-1 束外觀

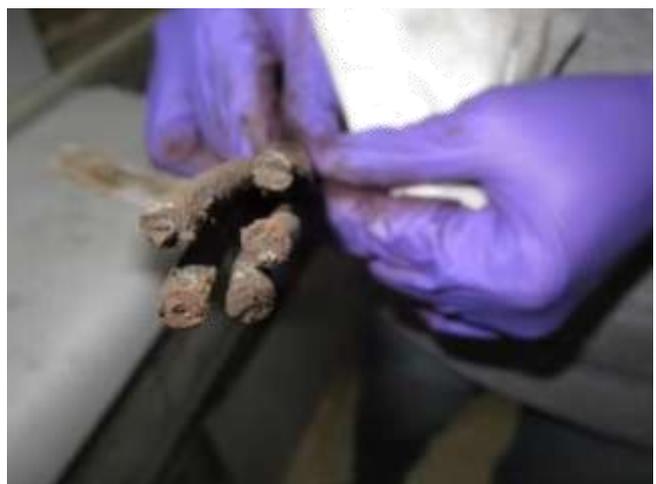


圖120 橋面 10 號錨頭殘留之鋼絞線 10-2 東外觀

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圖121 橋面 10 號錨頭殘留之鋼絞線 10-3 束外觀



圖122 橋面 10 號錨頭殘留之鋼絞線 10-4 束外觀



圖123 橋面 10 號錨頭殘留之鋼絞線 10-5 束外觀



圖124 橋面 10 號錨頭殘留之鋼絞線 10-6 束外觀

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圖125 橋面 10 號錨頭殘留之鋼絞線 10-7 束外觀



圖126 橋面 10 號錨頭殘留之鋼絞線 10-8 束外觀

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圖127 橋面 10 號錨頭殘留之鋼絞線 10-9 束外觀



圖128 橋面 10 號錨頭殘留之鋼絞線 10-10 束外觀



圖129 橋面 10 號錨頭殘留之鋼絞線 10-11 束外觀



圖130 橋面 11 號錨頭殘留之全部鋼絞線外觀

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圖131 橋面 11 號錨頭殘留之鋼絞線 11-1 束外觀

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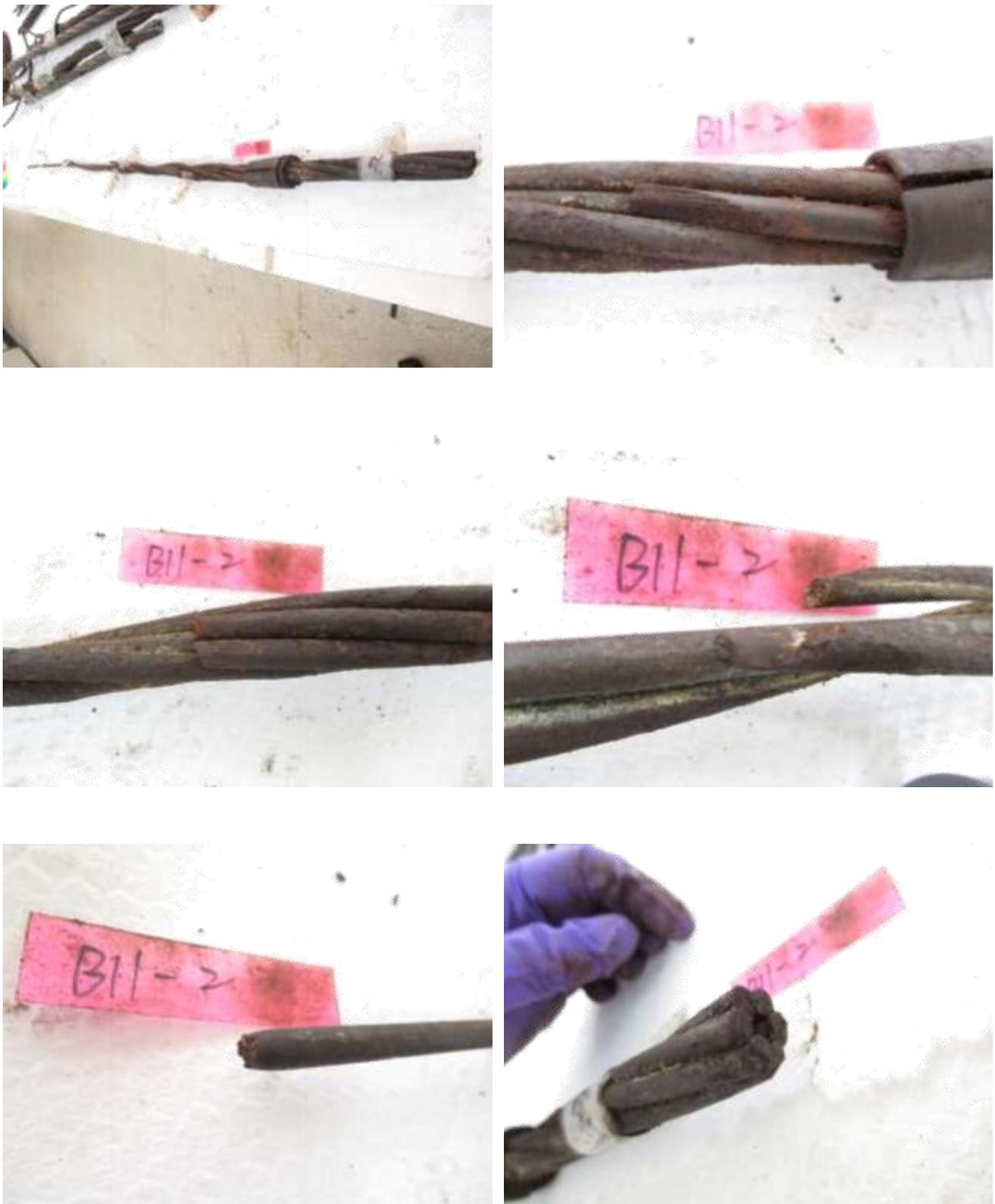


圖132 橋面 11 號錨頭殘留之鋼絞線 11-2 束外觀

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圖133 橋面 11 號錨頭殘留之鋼絞線 11-3 束外觀

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圖134 橋面 11 號錨頭殘留之鋼絞線 11-4 束外觀



圖135 橋面 11 號錨頭殘留之鋼絞線 11-5 束外觀

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圖136 橋面 11 號錨頭殘留之鋼絞線 11-6 束外觀



圖137 橋面 11 號錨頭殘留之鋼絞線 11-7 束外觀

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圖138 橋面 11 號錨頭殘留之鋼絞線 11-8 束外觀



圖139 橋面 11 號錨頭殘留之鋼絞線 11-9 束外觀



圖140 橋面 11 號錨頭殘留之鋼絞線 11-10 束外觀



圖141 橋面 12 號錨頭殘留之全部鋼絞線外觀



圖142 橋面 12 號錨頭殘留之鋼絞線 12-1 東外觀



圖143 橋面 12 號錨頭殘留之鋼絞線 12-2 束外觀

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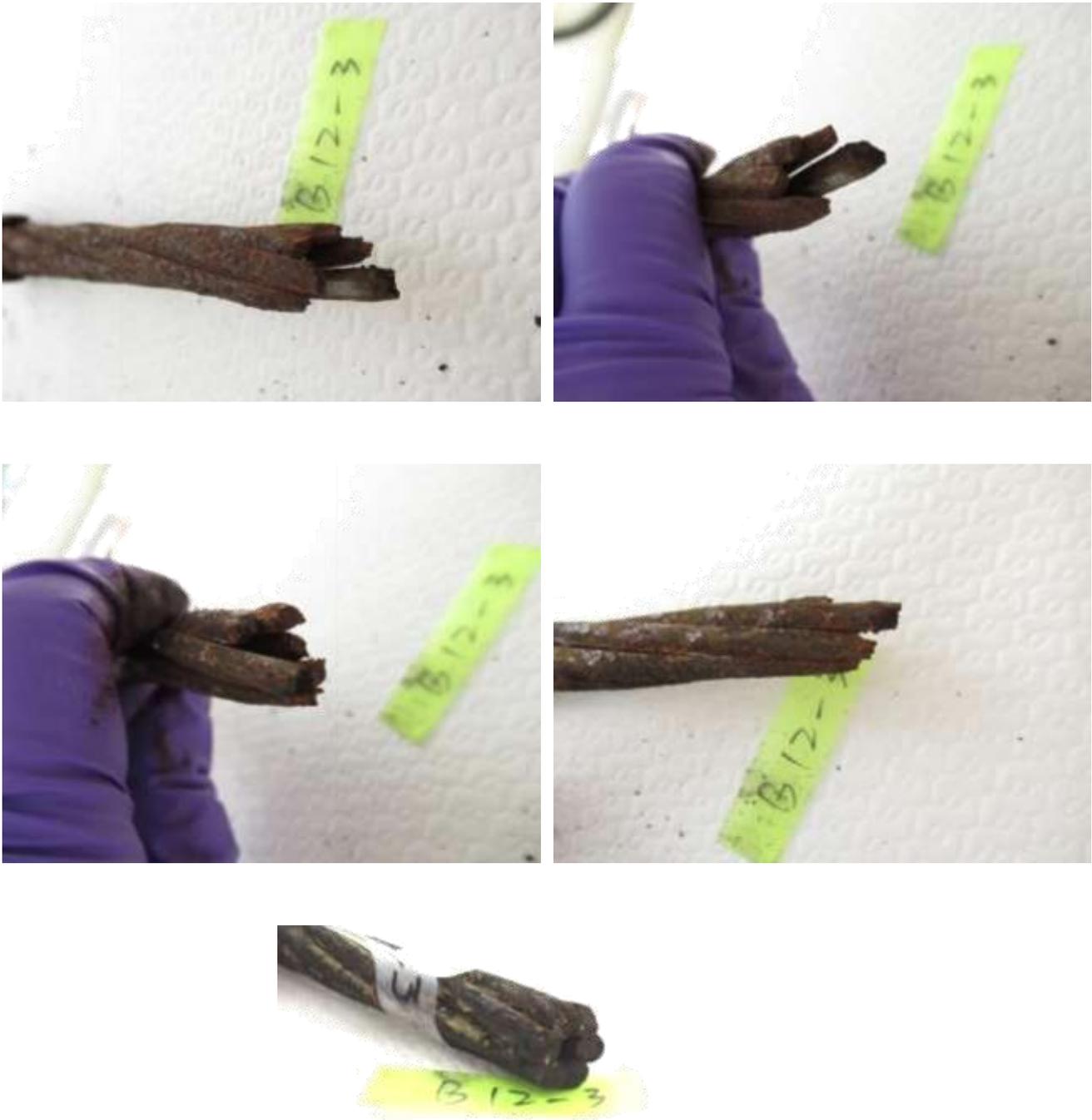


圖144 橋面 12 號錨頭殘留之鋼絞線 12-3 束外觀

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圖145 橋面 12 號錨頭殘留之鋼絞線 12-4 束外觀

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圖146 橋面 12 號錨頭殘留之鋼絞線 12-5 東外觀

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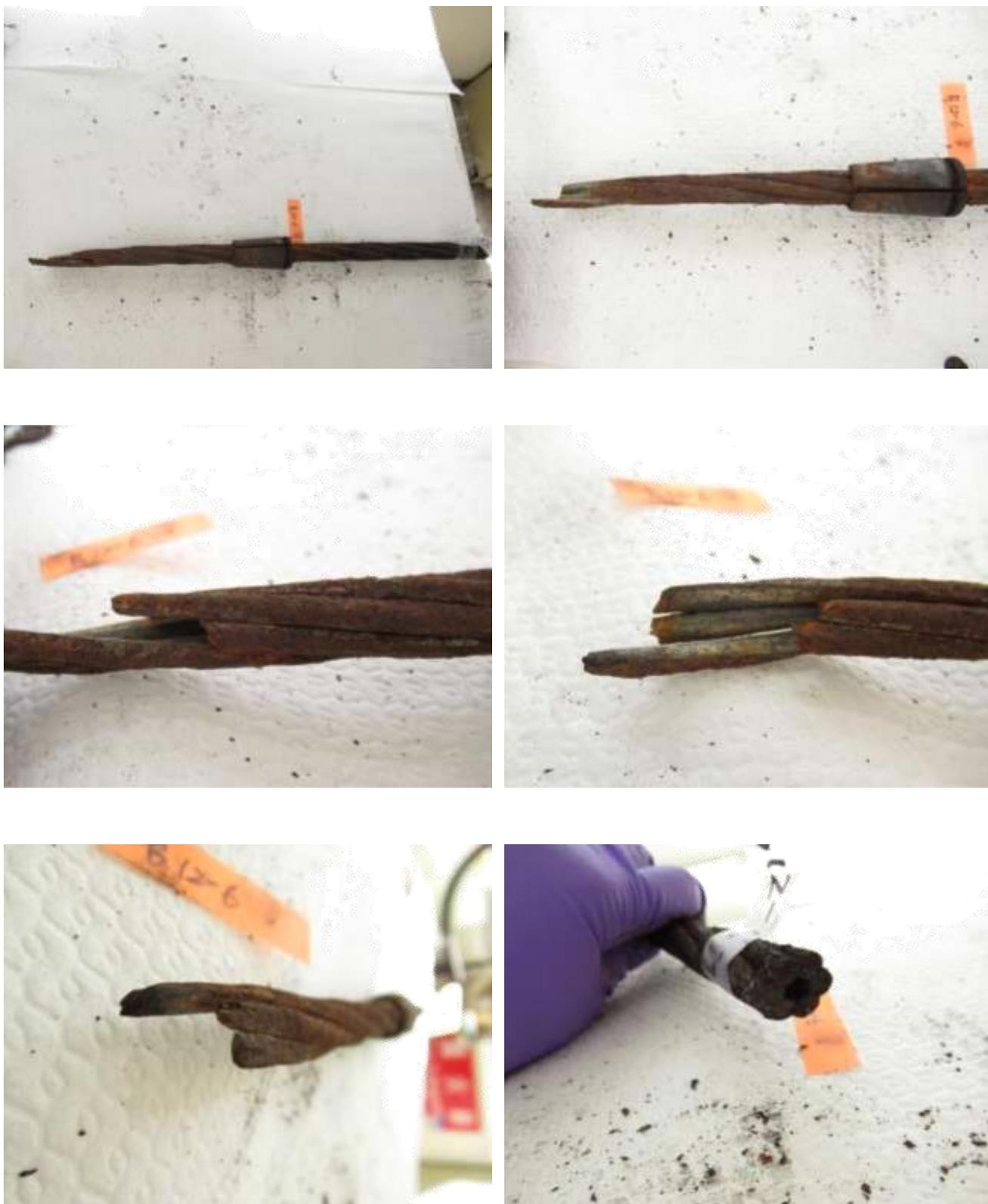


圖147 橋面 12 號錨頭殘留之鋼絞線 12-6 束外觀

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圖148 橋面 12 號錨頭殘留之鋼絞線 12-7 束外觀

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圖149 橋面 12 號錨頭殘留之鋼絞線 12-8 束外觀

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圖150 橋面 12 號錨頭殘留之鋼絞線 12-9 束外觀

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圖151 橋面 12 號錨頭殘留之鋼絞線 12-10 束外觀

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圖152 橋面 12 號錨頭殘留之鋼絞線 12-11 束外觀

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圖153 橋面 12 號錨頭殘留之鋼絞線 12-12 束外觀

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圖154 橋面 12 號錨頭殘留之鋼絞線 12-13 束外觀

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圖155 橋面 13 號錨頭殘留之全部鋼絞線外觀



圖156 橋面 13 號錨頭殘留之鋼絞線 13-1 束外觀



圖157 橋面 13 號錨頭殘留之鋼絞線 13-2 束外觀



圖158 橋面 13 號錨頭殘留之鋼絞線 13-3 東外觀

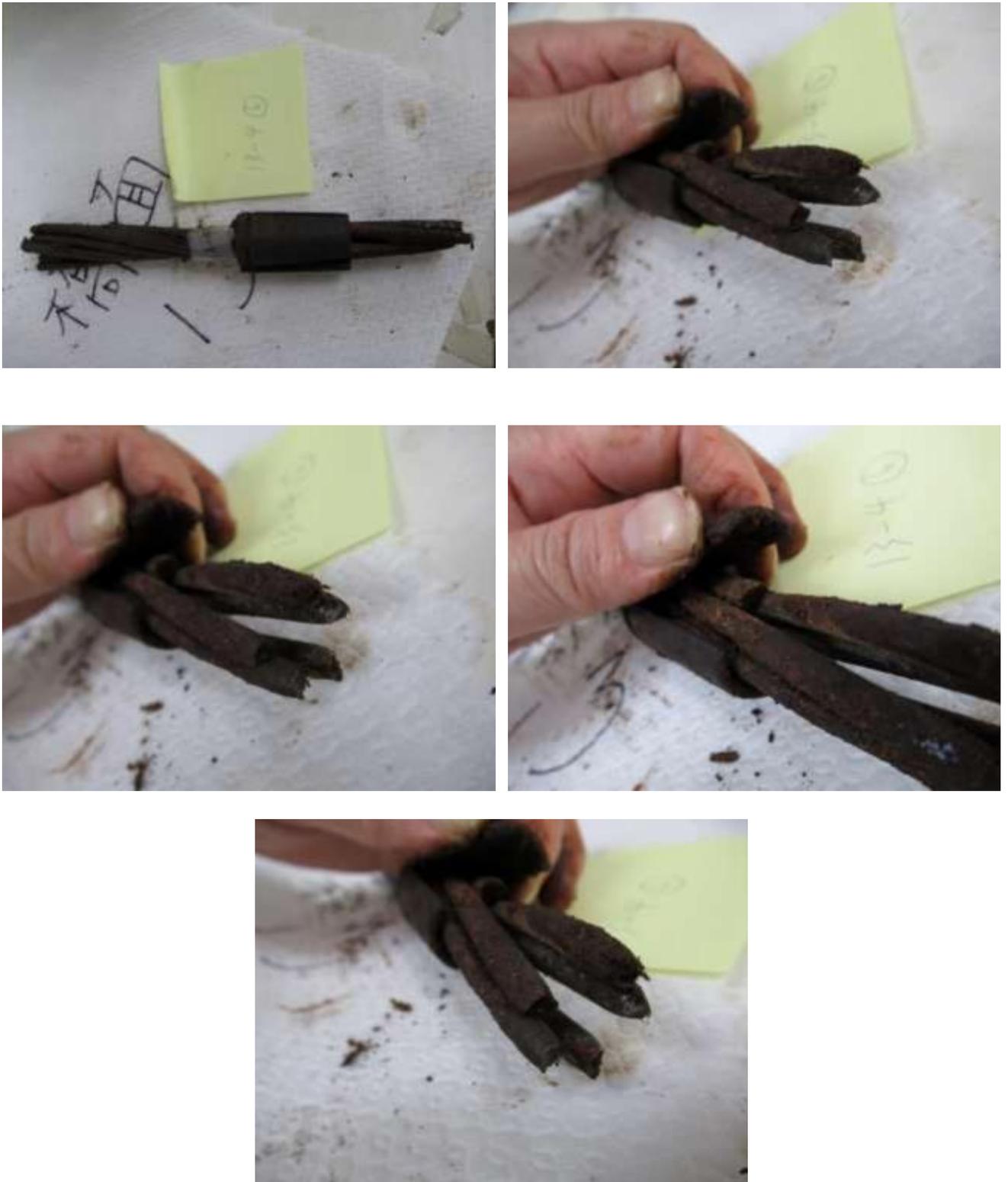


圖159 橋面 13 號錨頭殘留之鋼絞線 13-4 束外觀

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圖160 橋面 13 號錨頭殘留之鋼絞線 13-5 束外觀

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圖161 橋面 13 號錨頭殘留之鋼絞線 13-6 束外觀



圖162 橋面 13 號錨頭殘留之鋼絞線 13-7 束外觀

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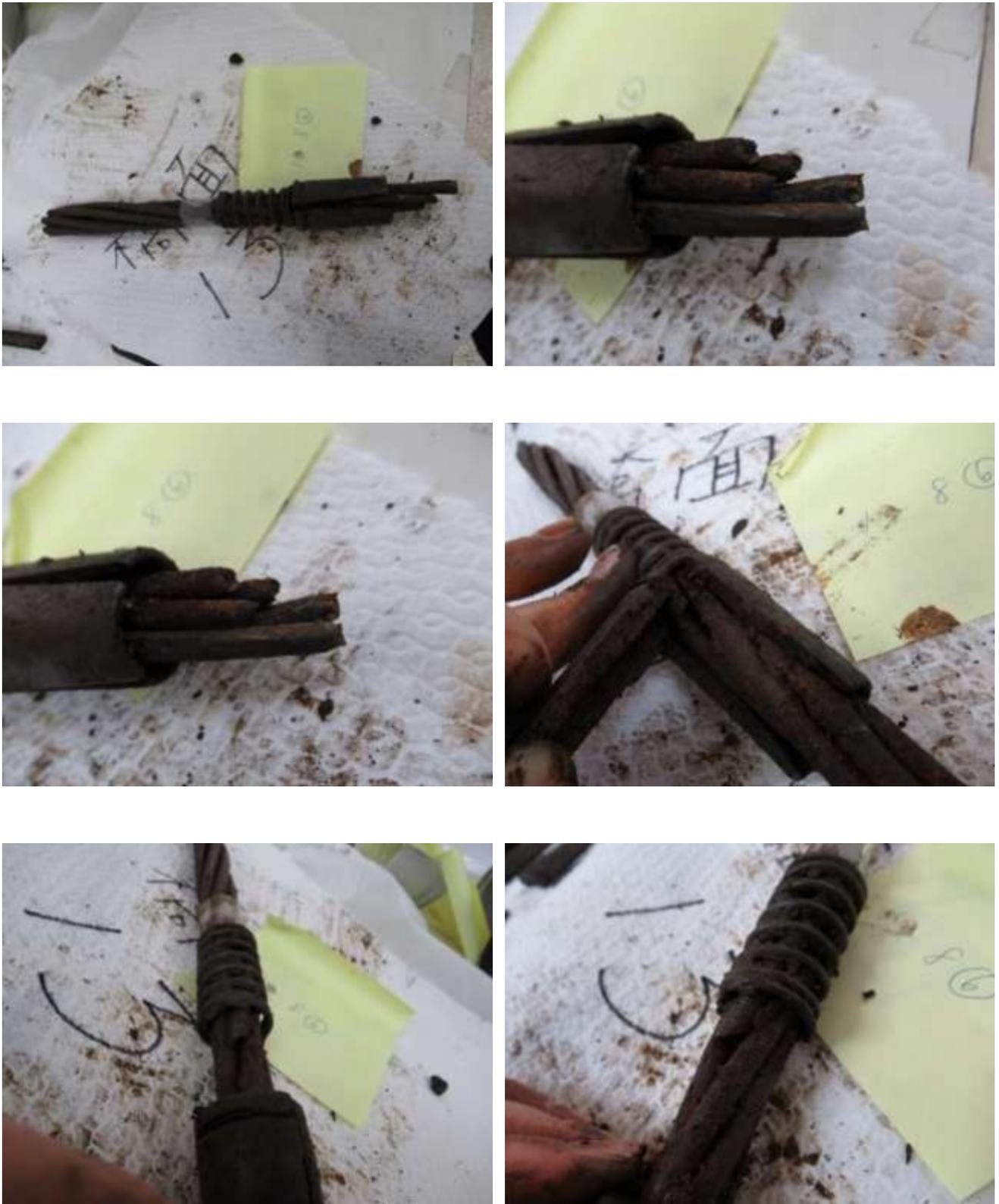


圖163 橋面 13 號錨頭殘留之鋼絞線 13-8 束外觀

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圖164 橋面 13 號錨頭殘留之鋼絞線 13-9 東外觀



圖165 橋面 13 號錨頭殘留之鋼絞線 13-10 束外觀



圖166 橋面 13 號錨頭殘留之鋼絞線 13-11 束外觀

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圖167 橋面 13 號錨頭殘留之鋼絞線 13-12 束外觀



圖168 橋面 13 號錨頭殘留之鋼絞線 13-13 束外觀

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圖169 橋面 13 號錨頭殘留之鋼絞線 13-14 束外觀

### 十七、 綜合討論

由前述各項分析及觀察結果，針對本案錨頭及鋼絞線失效原因重點討論如下：

1. 抽樣檢測的錨頭，其材質皆相同，機械性質皆相近，顯示可能為同商源或同批次生產之材料，而機械性質測試顯示該材質並未熱處理至最大強度。在鋼絞線材方面，抽樣測試結果顯示不同線束材質及表面鍍層狀態皆有差異，顯示鋼線材可能有不同來源，但機械性質測試顯示不同線束之強度及硬度差異不大，惟表面鍍鋅層有無將會影響線材本身之耐蝕性。
2. 針對事故發生後，上拱端及橋面端錨頭狀態彙整於表 10，排除孔位束不同的 1 和 13 號，上拱端錨頭僅編號 7、9、10、11、12 未破裂，其中上拱 10 號及 12 號錨頭未破裂且保留完整 13 束鋼絞線。由成份及機械性質分析可知錨頭使用之材質皆相同且強度硬度皆相似，但錨頭破裂之情況幾乎都發生在上拱端，除可能與鋼絞線狀態有關外，橋面端錨頭尚有承壓環包覆於外(橋面 1 號錨頭之承壓環並未包覆錨頭，如圖 59 所見)，故即使承受相同應力狀態亦較不易破裂。此外，一般而言發生失效時應是鋼纜斷裂而非錨頭破裂，但本案上拱端錨頭卻超過半數都破裂，且以原有鋼纜與錨頭搭配的拉伸測試(上拱 10 號錨頭)，亦發生鋼纜未斷而錨頭破裂之情形，顯示本案中錨頭材質強度與孔位設計的搭配並不足以承受全部 13 束或 17 束鋼絞線之最大強度拉力。然而，本案卻仍有編號 7、9、10、11、12 號之上下錨頭皆未破裂，在各編號之錨頭與各鋼線機械性質皆相近的情況下，顯然尚有其他與失效有關之因子存在。

表10 全部錨頭狀態統計表

	1	2	3	4	5	6	7	8	9	10	11	12	13
上拱	X	X	X	X	X	X	○	X	○	◎	○	◎	X
橋面	X	○	○	○	○	○	○	○	◎	○	○	◎	○

X：錨頭破裂    ○：錨頭未破    ◎：錨頭未破且殘留全部鋼絞線束頭

3. 如前述，由上拱錨頭破裂情況及上拱 10 號錨頭拉伸測試結果，可推知錨頭材質強度及孔位設計並無法承受全部 13 束(以 2~12 號錨頭而言)鋼絞線同時施加大應力之狀況，而編號 7、9、10、11、12 號之上下錨頭皆未破裂，故可合理推論這些編號的鋼纜在事故發生前已有部份線束斷裂，導致事故發生時僅有部份的線束對錨頭施力，剩餘鋼線束總強度已低於錨頭強度，故是線束被拉斷而錨頭未破裂。若要證實此論點，需尋找「部份鋼線束在事故發生前已斷裂」之事證。由於鋼絞線在成份分析及表面鍍層雖顯示線材有不同來源，但機械性質卻差異不大，故若要發生部份鋼線在事故前即已斷裂，鋼線材質並非主要原因，否則也不會使用超過 20 年才發生斷橋事故；而在線材檢視時發現許多鋼線皆有生鏽之情況，雖大部份鋼線皆有鍍鋅層保護，但鍍層分析亦顯示鍍鋅層消耗嚴重，局部區域外在環境的確有可能造成局部腐蝕嚴重，因此「生鏽」可能才是部份鋼線先斷裂之主因。事實上，由橋面端錨頭殘留之鋼線(如圖 98~圖 169)即可發現多數鋼線生鏽嚴重，而上拱端檢視的第 8、9、12 股鋼線(圖 21~圖 48)亦可發現多數鋼線在斷裂處皆有生鏽情況，因此生鏽除了可能是造成鋼線在事故前已發生斷裂之原因外，「生鏽程度」亦是在事故發生後，用以判斷鋼線斷裂時間點先後的重要依據。
4. 雖然許多鋼線由斷裂處外觀檢視即可推論橋面端生鏽較嚴重，為了更明確生鏽斷裂並非在短期間內發生，因此針對鋼線斷裂處上下端進一步分析比對，以獲得更多資訊。由於事故發生後，鋼纜斷裂且部份脫落，很難將斷裂處上下端的鋼線束對應起來，故選擇編號第 12 股的鋼纜作為分析標的，因僅有第 12 股是上下端錨頭皆殘留完整 13 束鋼絞線，較有機會比對出同一線束之上下端。經比對上拱第 12 股各鋼絞線束斷面(圖 36~圖 48)

及橋面 12 號錨頭殘留各線束外觀(圖 142~圖 154)，可發現上拱各線束之芯線皆為平整面，僅 12-10-7 及 12-11-7 兩芯線為拉伸破斷面，顯示 13 束中有 11 束之芯線被抽出，故末端仍保持平整面，而 12-10-7 及 12-11-7 兩芯線為拉伸破斷面，顯示另一端應留在橋面端線束中，而比對橋面 12 號錨頭殘留線束，可發現恰有 12-1 及 12-9 兩線束仍保有 7 線(圖 142及圖 150)，其餘 11 束皆為 6 線，故可確認上拱 12-10 及 12-11 兩線束對應到橋面 12-1 及 12-9 兩線束。此外，再比對此四線束，可發現橋面 12-1 束有兩鋼線長度較長(如圖 142)，而上拱 12-10 線束恰有兩線較短，故可確認此兩線束為同一鋼絞線束斷裂之兩端，如圖 170 中所見，因此此鋼絞線束最適合作為斷裂處上下端生鏽程度差異之比對。



圖170 橋面第 12-1 束與上拱 12-10 束對應外觀(左)及橋面第 12-1 束底部外觀(右)

5. 為了解生鏽與斷裂之時序，深入比對斷裂處兩端之生鏽程度差異。圖 171 為前述用以比對生鏽程度差異之鋼絞線束斷裂處兩端外觀比對照，可發現不論是較長的兩線或是其他較短的五線，橋面端的鋼線生鏽情況明顯較嚴重，較短的五線甚至已生鏽成蓬鬆狀而有層狀剝離的現象，而比對斷裂處兩端芯線的狀況亦是如此，如圖 172 所示，橋面端芯線一樣有較嚴重的生鏽情況。細部檢視橋面端 12-1 束鋼線鏽蝕情況如圖 173 所示，較長之兩鋼線在其他五線斷裂位置也有明顯鏽蝕情況，導致該處明顯變細，可推論在此處外在環境有較強的腐蝕因子，導致線材腐蝕嚴重。理論上，此處所在位置不應有太強的腐蝕環境存在，但由圖 59~圖 71 橋面錨頭拆解前外觀可知橋面錨頭周遭部件皆鏽蝕嚴重，除錨頭本身有生鏽情況外，多件螺紋調整管更嚴重鏽蝕，出現膨脹及層狀剝離情形，推論橋面端錨頭多數有積水情況，在水面與空氣交界處再加上海水鹽分作用，腐蝕環境較一般嚴重，長時間反應下才有此嚴重鏽蝕之情況。而同一線束斷裂處，橋面端與上拱端生鏽程度差異則意味著鋼線斷裂後，上拱端線材已較脫離嚴重腐蝕環境、橋面端線材則仍保持在該腐蝕環境中，因此若能確認橋面端線材斷裂處斷面鏽蝕深度，即可推測斷裂發生在事故發生當時或是更早以前即發生。



圖171 橋面第 12-1 束與上拱 12-10 束斷裂處上下端邊緣對應外觀



圖172 上拱 12-10 束芯線斷面外觀(左)及與橋面第 12-1 束芯線對應情形(右)



圖173 橋面第 12-1 束鏽蝕外觀

6. 為確認斷裂處兩端鋼線鏽蝕深度差異，擬進行線材剖面金相分析。先將橋面端 12-1 束進行拆解，如圖 174 所示，拆解後外觀則如圖 175 所見，可發現鋼線彼此包覆住之內部表面仍可看到殘留之鍍鋅層，故內部表面鏽蝕程度相對較輕微，尤其是芯線被邊線較完整包覆，因此除上端斷裂處鏽蝕較嚴重外，其他表面則有較多鍍鋅層或鋅層氧化殘留之形貌，如圖 176 所見。先將橋面端 12-1 束之芯線切取一適當長度，並另取一邊線線段，進行縱剖面鑲埋研磨拋光，而上拱端則取 12-10-7 芯線與 12-10-6 邊線，一樣進行縱剖面金相製備，再進行四段線材之剖面金相比對。圖 177 及圖 178 即為斷裂處上下端芯線與邊線縱剖面外觀照片，由剖面觀察可知不論是芯線或邊線，不論是上拱端或橋面端，斷裂處皆已有再生成相當厚度的鏽蝕層，且橋面端的鏽蝕層更厚，已達 mm 等級，已非新鮮鋼線斷裂後浸泡在海水中所能產生的氧化層厚度，故可確認此線束早在事故發生前早已斷裂，且由鏽蝕深度並非數週或數月可造成，應更早之前即已斷裂。



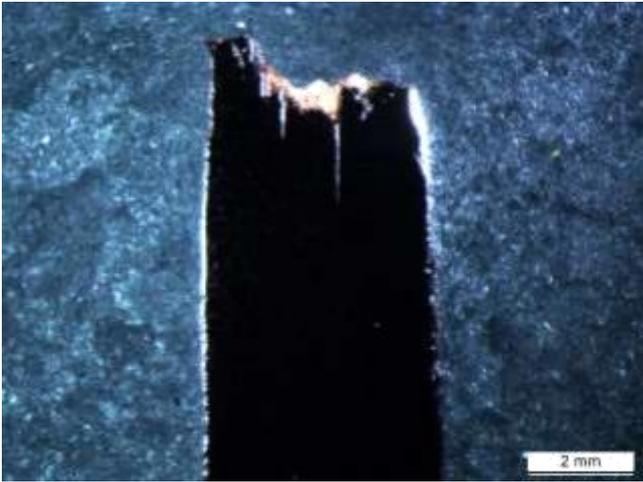
圖174 橋面第 12-1 束鋼線拆解過程



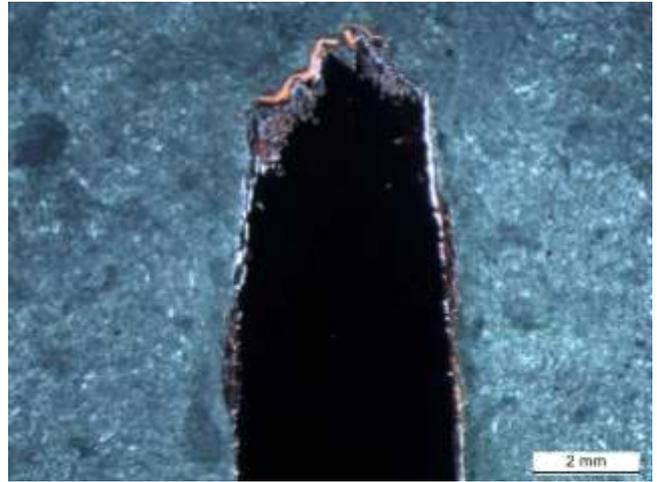
圖175 橋面第 12-1 束鋼線拆解後外觀



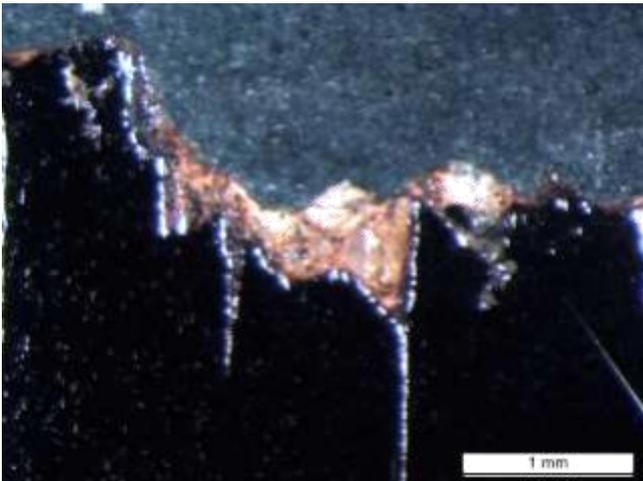
圖176 橋面第 12-1 束之芯線不同面外觀



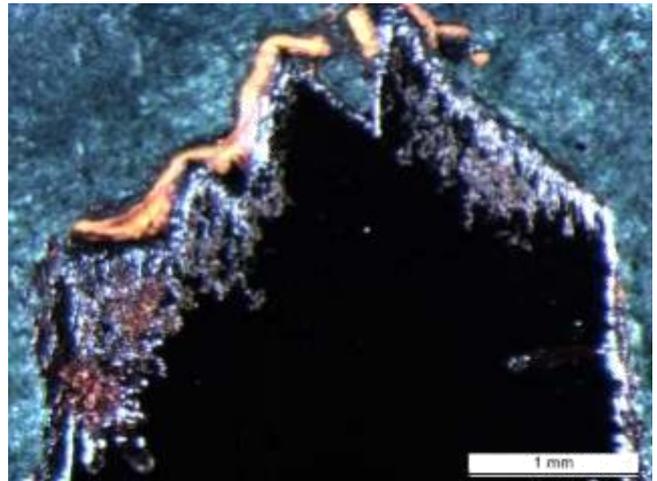
(a) 上拱 12-10-7 芯線低倍率



(b) 橋面 12-1 束芯線低倍率

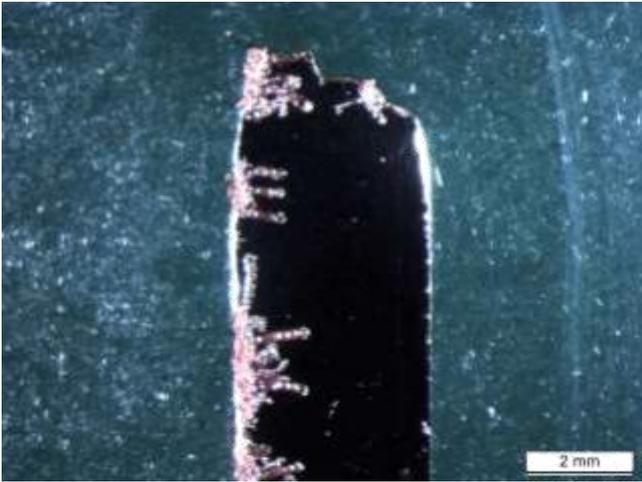


(c) 上拱 12-10-7 芯線高倍率

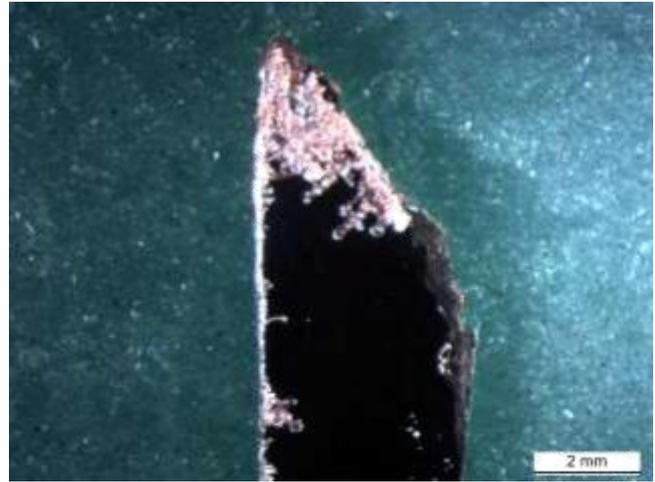


(d) 橋面 12-1 束芯線高倍率

圖177 第12股其中一束鋼絞線之芯線斷裂處上下端剖面實體顯微鏡觀察照片



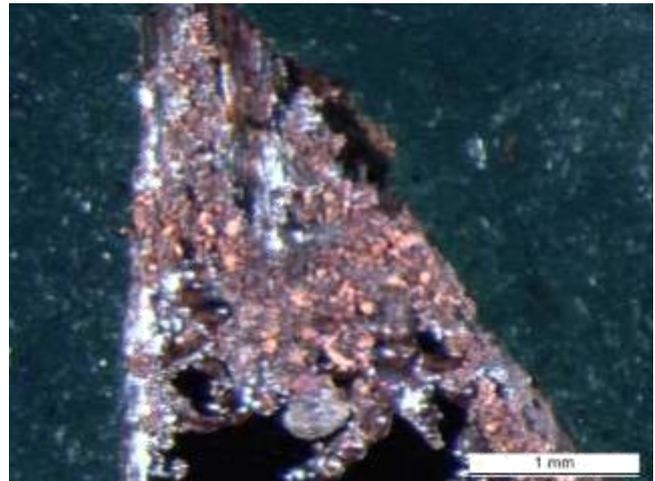
(a) 上拱 12-10-6 邊線低倍率



(b) 橋面 12-1 束其一邊線低倍率



(c) 上拱 12-10-6 邊線高倍率



(d) 橋面 12-1 束其一邊線高倍率

圖178 第 12 股其中一束鋼絞線之邊線斷裂處上下端(非同一線)剖面實體顯微鏡觀察照片

7. 此橋面 12-1 束鋼絞線尚可獲得更多訊息，由於其有兩線較長，而較長之兩線在其他五線斷裂處亦有嚴重鏽蝕情形，如圖 179 所示，因此可推論此束鋼絞線之七鋼線並非同一時期斷裂。經量測較長兩線之中段腐蝕嚴重位置，如圖 180，該處肉厚僅剩約 3 mm，寬度僅剩約 4 mm，亦即剩餘的線材已不到原本線徑的三分之一，應已無法承受正常之載重，但卻未從此處斷裂，顯然是因為上方已先斷裂，故此處已不再承受應力，故可知此二線為七線中最先斷裂。由圖 170 可知此二線斷裂處與另五線斷裂處之間之鋼線表面皆有鏽蝕情況，可推論原此橋面錨頭積水深度可能達兩線斷裂位置，此二線在該處先發生鏽蝕，而在應力作用下加速了應力腐蝕，導致此二線先斷裂，而後積水深度逐漸降低，故此過程中線材表面皆持續有生鏽情形，而降至五線斷裂處時維持了較長的時間(或水面在此深度附近反覆升降)，導致此處鏽蝕較嚴重，而未斷的五線因仍有應力作用故腐蝕較快終至斷裂，較長的兩線無應力加速腐蝕，故僅有較嚴重鏽蝕而未斷裂。



圖179 橋面第 12-1 束之兩條較長邊線鏽蝕中段外觀



圖180 橋面第 12-1 束之兩條較長邊線鏽蝕中段肉厚及寬度量測情形

8. 由以上各點之討論，可確認有部份鋼絞線束或鋼線，在事故發生前早已因鏽蝕及應力加速腐蝕作用而斷裂，同一錨頭的線束在部份線束已斷裂的情況下，將承受更大的應力，若外在腐蝕環境並未消除，則將更加速應力腐蝕導致斷裂。因此雖然錨頭材質強度及孔位設計無法承受全部線束總強度的應力，但若有部份鋼絞線束因鏽蝕而斷裂的錨頭，只要殘餘線束總強度小於錨頭孔位設計強度，錨頭就不會因應力過大而破裂，反之，只要錨頭上未斷線束超過一定數量或全部未斷，則事故發生時突來的大應力將導致上拱錨頭破裂，也因此本案有部份上拱錨頭破裂、部份卻無破裂之情況。雖然錨頭的材質強度和孔位設計搭配不足以負荷全部線束的總強度，但在全部鋼絞線未斷的情況下，應力平均分配，並不會發生錨頭破裂的情況，因此，本案事故的主因仍是鏽蝕導致鋼絞線斷裂所導致，當斷裂的線束逐漸增加，剩餘未斷的線束及其所在的錨頭承受應力亦逐漸增加，當應力大於錨頭所能承受之強度，即會發生破裂，而一旦有一錨頭破裂，其原本所負載的應力將再分配至其他未斷線材，因而引發連鎖反應，造成錨頭連續破裂而造成斷橋事故。

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## 附錄 5 法西奈回復文件



3 April 2020

Our ref : 20/0371/MM

Taiwan Transportation Safety Board  
11<sup>th</sup> Floor,  
200, Section 3, Beixin Road,  
Xindian District,  
New Taipei City 231, TAIWAN (R.O.C.)  
Attn : Mr. Thomas Wang – Chief Investigator (Investigator in Charge)

Dear Sir,

Thank you for your letter dated March 16, 2020.

Please find below answers to your queries based on information we could collect from the nineties.

1. *The involvement of the Freyssinet with the Nanfang'ao Bridge, please provide the supporting documents if available.*

Reply: From our records and questioning of Staff (still active, and some retirees); Freyssinet did not have any involvement in this bridge.

2. *Was there a Freyssinet sales office in Taipei between 1995 and 1998? The only channel to purchase Freyssinet anchorages was from your sales channels during that time?*

Reply: Freyssinet had a branch office in Taiwan which was created in 1995 and was closed around 2003. It was the sole representative of Freyssinet in Taiwan.

3. *How many Freyssinet Stay Cable systems (anchorages) were sold to Taiwan between 1990 and 1998. Please provide details information if available.*

Reply: Freyssinet provided the stay cable systems to two bridges in Taiwan: The Mei-shywe bridge in Taipei in 1994, sold through an agent (Pre-tech company), and the T-05 Bridge in Tainan in 1998, sold through our branch office. Please refer to the list of references Appendix A.

4. *Would Freyssinet provide on-site supports/instructions for the installation of the stay cables if Nanfang'ao Bridge were using Freyssinet products during that time?*

Reply: Freyssinet was not involved in this Nanfang'ao bridge.

### FREYSSINET Hong Kong Limited

Unit 705-6, 7/F, Tower I, Cheung Sha Wan Plaza,  
833 Cheung Sha Wan Road, Kowloon, Hong Kong.  
Tel : (852) 2794 0322 Fax : (852) 2338 3264  
E-mail : info@freysinet.com.hk



A SUBSIDIARY OF  SOLETANCHE FREYSSINET



Page 2  
Our ref : 20/0371/MM

5. How to identify Freyssinet anchorage during the time between 1995 and 1998?  
Please provide supporting documents/examples if available.

Reply: Freyssinet's Stay Cable anchorages did bear an identification number [FI xxx] on the face of the anchorage as shown on Appendix B.

6. Base on the photos attached with this mail, please identify whether the anchorages in the photos were Freyssinet Anchorages or not. Please provide the reason.

Reply: Whilst pictures are never 100% clear (a visual inspection is preferable and can be arranged when mobility restrictions will be lifted after the Covid-19 Pandemic).

From these pictures, we can see that it is NOT a Stay Cable system; please refer to appendix C Fig. 19 which shows the Freyssinet system having the following components:

- a. Bundle of strands inside an outer HDPE sheath
- b. Freyssinet monostrands composed of:
  - i. Galvanized strand
  - ii. Wax that fills the interstices between the wires
  - iii. High Density polyethylene sheath extruded onto each strand
- c. An Anchorage steel tube which is housing the stuffing box
- d. A stuffing box
- e. Plastic bushings at the inner face of the anchorage block
- f. Special stay cable jaws

We hope the above answers / clarifications are of assistance to you, but please do not hesitate to contact us for further details if required,

Yours faithfully,  
For and on behalf of  
**FREYSSINET Hong Kong Limited**



**FREYSSINET Hong Kong Limited**

Unit 705-6, 7/F, Tower I, Cheung Sha Wan Plaza,  
833 Cheung Sha Wan Road, Kowloon, Hong Kong.  
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## **Appendix A**

## b. référence

les références ci-dessous concernent des projets pour lesquels des sociétés du Groupe Freyssinet sont intervenues en qualité de concepteur, de fournisseur ou spécialiste de haubans.

Mise en Service	Nom	Pays	Usage	Tablier	Haubannage	Travée principale (mètre)	Protection anti-corrosion (partie courante)
1977	Brotonne	France	Routier	Béton		320	Coulis de ciment/tube métallique
1977	Pasco-Kennewick	Etats-Unis	Routier	Béton		299	Coulis de ciment/tube en PEHD
1978	Kwang Fu	Taiwan	Routier	Béton		134	Coulis de ciment/tube en PEHD
1978	Rande	Espagne	Routier	Acier		400	Coulis de ciment/tube métallique
1978	Meylan	France	Piétonnier	Béton		79	Coulis de ciment/tube en PEHD
1979	Illhof	France	Piétonnier	Béton		64	Coulis de ciment/tube en PEHD
1980	Cotton Tree Drive	Hong Kong	Piétonnier	Béton		60	Coulis de ciment/tube en PEHD
1980	Nishikigaoka	Japon	Routier	Béton		52	Coulis de ciment/tube en PEHD
1982	Luling	Etats-Unis	Routier	Acier		373	Coulis de ciment/tube en acier
1983	Barrios de Luna	Espagne	Routier	Béton		440	Coulis de ciment/tube en PEHD
1983	Coatzacoalcos	Mexique	Routier	Béton		288	Coulis de ciment/tube métallique
1984	Cathédrale d'Abidjan	Côte d'Ivoire	-	-	-	-	Coulis de ciment/tube en PEHD
1985	Sambre	Belgique	Ferroviaire	Mixte		106	Monotoron Freyssinet
1988	Tampico	Mexique	Routier	Béton/Acier		360	Toron galvanisé/gaine en PEHD/cire
1988	Ville-sur-Haine	Belgique	Piétonnier	Acier		70	Monotoron Freyssinet
1988	Pertuiset	France	Routier	Béton		132	Monotoron Freyssinet
1988	Les Gures	France	Routier	Béton		30	Monotoron Freyssinet/tube en PEHD
1989	Wandre	Belgique	Routier	Béton		168	Monotoron Freyssinet/tube en acier
1989	Marbella	Espagne	Routier	Béton		72	Monotoron Freyssinet/tube en acier
1989	Grand Tressan	France	Piétonnier	Béton		37	Monotoron Freyssinet
1990	Kamizuma	Japon	Routier	Béton		103	Coulis de ciment/tube en PEHD
1991	Rajo Gomez	Mexique	Routier	Acier		53	Monotoron Freyssinet
1991	Republica Federal	Mexique	Routier	Acier		50	Monotoron Freyssinet
1991	Crisóstomo Bonilla	Mexique	Routier	Acier		50	Monotoron Freyssinet
1991	Canal San Juan	Mexique	Routier	Acier		55	Monotoron Freyssinet
1991	Guadiano	Espagne/Portugal	Routier	Béton		324	Monotoron Freyssinet
1991	Arade	Portugal	Routier	Béton		256	Monotoron Freyssinet
1991	La Bidouze	France	Routier	Béton		67	Monotoron Freyssinet/tube en PEHD
1991	Les Martyrs	France	Routier	Béton		120	Monotoron Freyssinet
1991	Labofina	Belgique	Routier	Acier		60	Monotoron Freyssinet/tube en acier
1991	Shiraya	Japon	Routier	Béton		124	Coulis de ciment/tube en FRP
1992	Twin Harp	Japon	Routier	Béton		140	Coulis de ciment/tube en PEHD
1992	Aomori	Japon	Routier	Béton		240	Coulis de ciment/tube en FRP
1992	Bourgogne	France	Routier	Béton		152	Monotoron Freyssinet
1992	Pas du Lac	France	Routier	Béton		68	Monotoron Freyssinet
1992	PS de Beaune	France	Piétonnier	Acier		35	Toron galvanisé/gaine en PEHD/cire
1992	Poplar	France	Routier	Béton		45	Monotoron Freyssinet
1992	La Barqueta	Royaume-Uni	Piétonnier	Acier		45	Monotoron Freyssinet/tube en acier
1992	Collserola	Espagne	Routier	Acier		168	Monotoron Freyssinet/tube en acier
1992	Forges Alichamps	Espagne	Telecom	Tour	-	-	Monotoron Freyssinet
1993	Ibor	France	Routier	Béton		59	Coulis de ciment/tube en PEHD
1993	Itaruta	Espagne	Routier	Béton/Acier		42	Monotoron Freyssinet
1993	Pesmont	Japon	Routier	Béton		108	Coulis de ciment/tube en FRP
1993	Aracataca	Australie	Piétonnier	Acier		42	Monotoron Freyssinet
1993	Estancia el Zapote	Mexique	Routier	Mixte		312	Monotoron Freyssinet/tube en PEHD
1993		Mexique	Routier	Mixte		176	Toron/résine époxy/tube en PEHD

Mise en Service	Nom	Pays	Usage	Tablier	Haubannage	Travée principale (mètre)	Protection anti-corrosion (partie courante)
1993	La Arena	Espagne	Routier	Mixte		105	Monotoron Freyssinet
1993	Colindres	Espagne	Routier	Béton		125	Monotoron Freyssinet/tube en acier
1993	Alende	Espagne	Routier	Béton		42	Monotoron Freyssinet/tube en acier
1994	Macau-Taipa	Macao	Routier	Béton		112	Monotoron Freyssinet
1994	Tamaraceite	Espagne	Routier	Béton		162	Monotoron Freyssinet/tube en PEHD
1994	Alzette	Luxembourg	Routier	Mixte		130	Monotoron Freyssinet/tube en acier
1994	Iroise	France	Routier	Béton		400	Toron galvanisé/gaine en PEHD/cire
1994	Rw 16 Rotterdam	Pays-Bas	Piétonnier	Acier		91	Monotoron Freyssinet/tube en PEHD
1994	L'Humanité	Belgique	Routier	Mixte		61	Monotoron Freyssinet/tube en acier
1994	Glebe Island	Australie	Routier	Béton		345	Monotoron Freyssinet/tube en PEHD
1995	Zaltbommel	Pays-Bas	Routier	Béton		256	Monotoron Freyssinet/tube en PEHD
1995	Normandie	France	Routier	Béton/Acier		856	Monotoron Freyss./carénage en 2 demi-coquilles
1995	Lerez	Espagne	Routier	Béton		124	Monotoron Freyssinet/tube en PEHD
1996	Second Severn	Royaume-Uni	Routier	Acier		456	Toron galvanisé/gaine en PEHD/cire
1996	Mei-Shywe	Taiwan	Routier	Acier		210	Monotoron Freyssinet
1996	Seogang Bridge	Corée	Routier	Acier		150	Monotoron Freyssinet/tube en PEHD
1996	PS d'Orange	France	Routier	Acier		29	Monotoron Freyssinet/tube en PEHD
1996	Concor East Overpass	Afrique du Sud	Routier	Mixte		49	Monotoron Freyssinet
1997	Pereira-Dosquebradas	Colombie	Routier	Mixte		211	Monotoron Freyssinet/tube en PEHD
1997	Karkistensalmi	Finlande	Routier	Acier		240	Monotoron Freyssinet/tube en PEHD
1997	Dee Crossing	Royaume-Uni	Routier	Béton		194	Monotoron Freyssinet/tube en PEHD/DGD
1997	PS N° 2, 4 et 9/A2	Portugal	Routier	Béton		64	Monotoron Freyssinet
1997	Izmir-Güzelyali	Turquie	Piétonnier	Mixte		56	Monotoron Freyssinet
1997	Deux PS à Malpensa 2000	Italie	Routier	Béton		70	Monotoron Freyssinet/tube en PEHD
1997	Oglio	Italie	Routier	Mixte		70	Monotoron Freyssinet/tube en PEHD
1997	Tenkenji	Japon	Routier	Béton		219	Coulis de ciment/tube en PEHD
1998	Oued Dib	Algérie	Routier	Béton		280	Monotoron Freyssinet/tube en PEHD
*	Général Belgrano	Argentine	Routier	Béton		245	Monotoron Freyssinet
*	Zárate-Brazo Largo	Argentine	Routier/Ferroviaire	Acier		330	Monotoron Freyssinet
**	Ting Kau	Hong Kong	Routier	Mixte		2 x 475	Monotoron Freyssinet/tube en PEHD
**	West Kowloon	Hong Kong	Piétonnier	Béton		120	Monotoron Freyssinet/tube en PEHD
**	Vasco de Gama	Portugal	Routier	Mixte		420	Monotoron Freyssinet/tube en PEHD/DGD
**	Val-Benoît	Belgique	Routier	Béton		162	Monotoron Freyssinet/tube en acier
**	Santarém	Portugal	Routier	Béton		246	Monotoron Freyssinet/tube en PEHD
**	Santo Tirso	Portugal	Routier	Béton		33	Monotoron Freyssinet/tube en PEHD
**	Indooroopilly	Australie	Piétonnier	Acier		180	Monotoron Freyssinet/tube en PEHD
**	Sydney Airport	Australie	Routier	Mixte		105	Monotoron Freyssinet/tube en PEHD
**	Pisuerga	Espagne	Routier	Béton		120	Monotoron Freyssinet
**	Archena	Espagne	Routier	Béton		75	Monotoron Freyssinet/tube en PEHD
**	Musée Allemand de la Technique/Berlin	Allemagne	-	-	-	-	Monotoron Freyssinet/tube en PEHD
**	Assouan	Egypte	Routier	Béton		250	Monotoron Freyssinet/tube en PEHD
**	My Thuan	Vietnam	Routier	Béton		350	Monotoron Freyssinet/tube en PEHD/DGD
**	Cape Girardeau	Etats-Unis	Routier	Acier		350	Monotoron gainé graissé/gaine PEHD/coulis de ciment
**	Tarascon-Beaucaire	France	Routier	Béton		193	Monotoron Freyssinet/tube en PEHD
**	Øresund	Danemark/Suède	Routier/Ferroviaire	Mixte		490	Monotoron Freyssinet/tube en PEHD/DGD

\* Remplacement des haubans.  
 \*\* En construction

## **Appendix B**

GLEBE ISLAND

ESSAI FATIGUE 56 H 15 / MUNCIH



TYPE

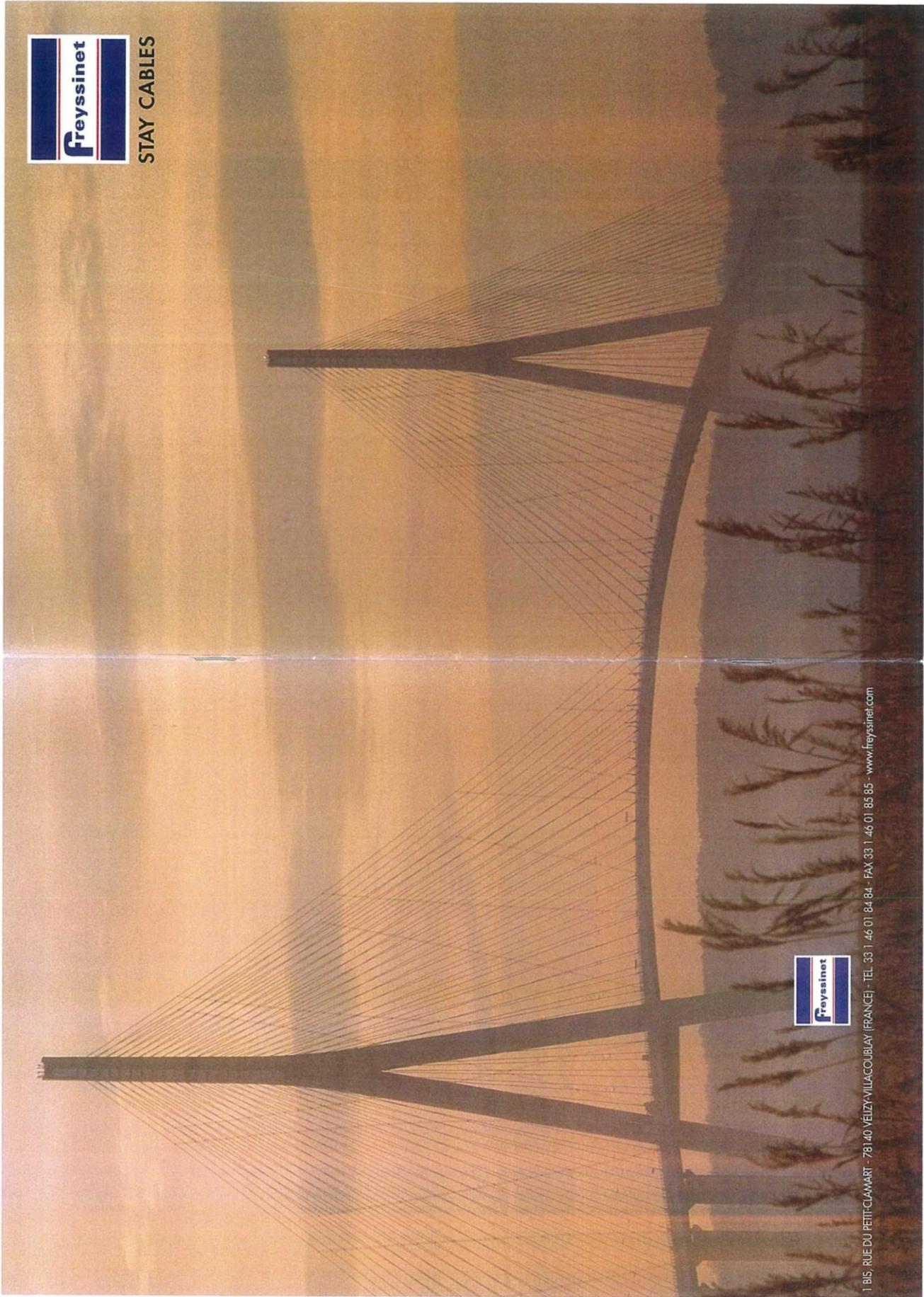
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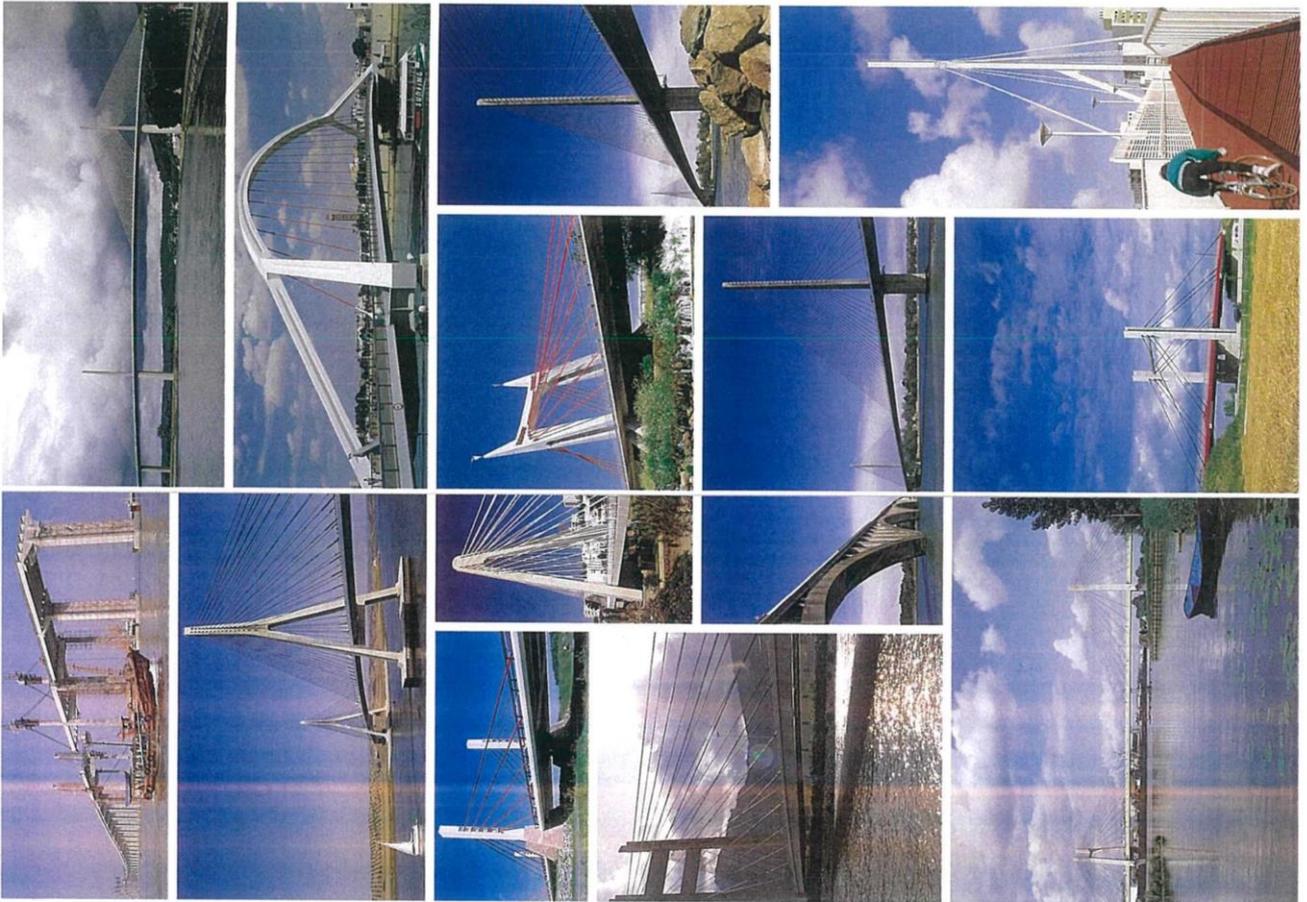
## **Appendix C**



STAY CABLES



1 BIS, RUE DU PETITCLAWART - 78140 VEIZY-VILACOURLAY (FRANCE) - TEL 33 1 46 01 84 84 - FAX 33 1 46 01 85 85 - www.freysinet.com



**freysinet**  
**STAY CABLES**  
SEPTEMBER 1999

# 1. introduction

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Over the past fifteen years, the rapid development of stayed structures, in particular cable-stayed bridges, has indicated a growing interest in this type of structure.

The use of stay cables in the construction of bridges of over 856 metre span, of which decks may be of steel or concrete, has extended this technique into an area which was previously the domain of suspension bridges.

Modern stay cable techniques, using cables and anchorages derived from prestressing technology, facilitate both the installation of stay cables and, when necessary, their rapid replacement.

Recent applications have demanded varied and, above all, developing specifications. The aim, therefore, of this brochure is to define the fundamental requirements of stay cable technology.

The Freyssinet stay cables described have been designed to satisfy the most widely varying demands and to cover a vast field of applications, of which bridges may be considered as being the most representative of stay cable technology.



Wandre bridge (Belgium)

## 2. historical aspects

It is generally considered that the origin of stay cables goes back to the Middle Ages; the drawbridges of the feudal castles, whose decks could be raised so as to prevent entry, were in fact stayed bridges.

It was not however until the 17th century that true cable-stayed bridges made their appearance; the first example being a structure designed in 1617 by a Venetian called Faustus Verantius. In the 18th century, this type of structure was developed considerably and, a century later, in 1817, the first steel cable-stayed bridge was constructed.

Unfortunately, as a result of accidents due to a lack of adequate methods of structural analysis, and of dimensioning and checking the stays, the development of the technique slowed down progressively until it was almost abandoned.

The two most spectacular accidents which occurred at that time were: the collapse of Dryburgh Abbey bridge [Scotland] in 1817, due to excessive loading as a result of oscillation caused by the wind, and, in 1824, the failure of the Scale bridge (Germany), due to overloading of the deck (fig. 2).

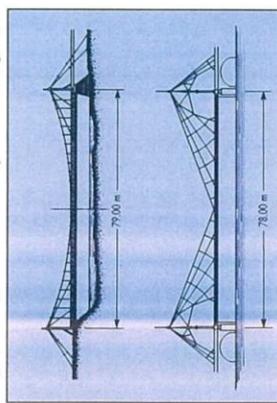


Fig. 2 - Dryburgh Abbey and Scale bridges

Following these accidents, the French designer Navier recommended the use of suspension bridges and very few cable-stayed bridges were constructed for more than one hundred years.

However, among the few stayed bridges constructed during this period were: Wheeling bridges over the Ohio river (1846) and the railway bridge over the Niagara river (1855) - both constructed by Roebling in the United States.

At the outset of the 20th century, Anodini, a French contractor, built two cable-stayed transporter bridges, one in Marseilles in 1903 (fig. 3), the other in Nantes in 1905.

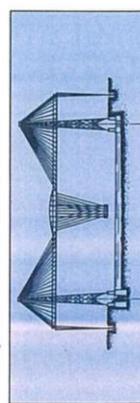


Fig. 3 - Transporter bridge at Marseilles (France)

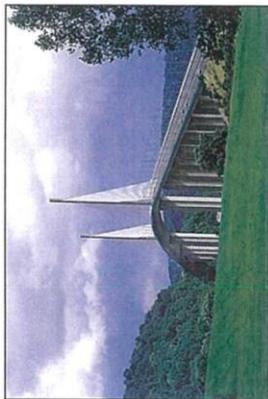


Fig. 4 - Brotonne bridge (France)

After 1945, the absolute necessity of reconstructing, in an economical manner, those bridges destroyed during the second world war, together with the progress achieved in the field of steel construction, brought about the revival of cable-stayed bridges.

Renowned engineers, such as Dischinger, proved the superiority, both technical and economical, of such structures as compared with suspension bridges for a span range of one hundred to several hundred metres.

The first of this new generation was Sjörsund bridge in Sweden, built in 1955.

This was followed by several other bridges, among which Brotonne in France (1977) (fig. 4), Rance (1978) and Barrios de Luna (1983) in Spain, Coatzacoalcas (1984) and Tampico (1988) in Mexico (fig. 5), are worthy of note.

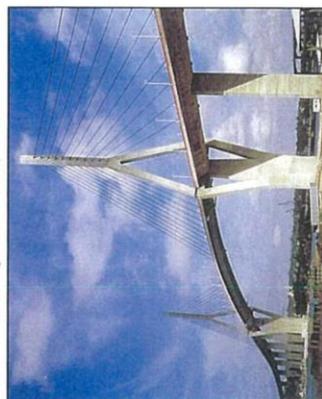


Fig. 5 - Tampico bridge (Mexico)

The main improvements made since then were the optimisation of stay distribution in order to diminish local stresses in the structure and the simplification and rapidity of replacement of a stay in case of damage, without appreciably upsetting the prearranged static functioning of the structure.

The span length of cable-stayed bridges, either with steel, concrete or composite decks, has increased very rapidly, to attain values (850 m in the Normandie bridge), which were unthinkable ten years ago.

## 3. function and qualities of a stay cable

### 3.1 Functioning

By virtue of its structure and installation, a stay cable is compared to and sometimes confused with a prestressing cable. However, although technologically closely related, their nature and functioning are quite different.

Comparison of a prestressed concrete bridge, built in cantilever of precast segments, with a cable-stayed bridge, using segments in a similar manner, illustrates the fundamental differences between the prestressing cable and the stay cable (fig. 6).

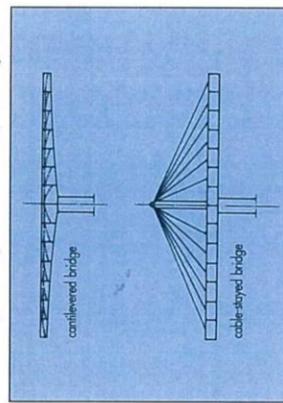


Fig. 6 - Segmental bridges

In the case of the cantilevered bridge, the prestressing cables exert an active force on the deck. Due to the very great stiffness of the deck as compared with that of the cables, the latter may be highly stressed, independently of the applied loads, with due regard to the strength of the deck.

Subsequent stress variations in the cables result from those in the surrounding concrete and are balanced in the proportion of the modulus of elasticity of the two materials: steel and concrete. If one assumes, for example, a modular ratio  $m = 5$ , a stress range of  $10 \text{ N/mm}^2$  in the concrete, under the action of the live loads, corresponds to a stress range of  $50 \text{ N/mm}^2$  in the steel.

In a cable-stayed bridge, the cables are tensioned elements of a triangulated structure, formed by the deck, the pylon and the stay cables. The latter are passive hangers, although adjustable, and may not be tensioned to a value greater than that of the loads applied to the node, the intersection of the deck with the stay, without causing deformations which are incompatible with the actual strength of the deck (fig. 7).

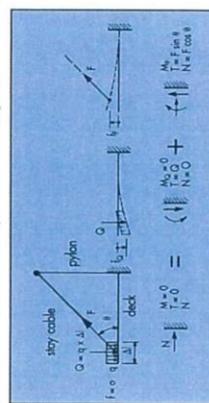


Fig. 7 - Triangulated structure, formed by the deck, the pylon and the stay cable

In this configuration, the deck is analysed as a continuous beam over multiple elastic supports which are provided by the stay cables. Adjustment of the tension in the stay cables provides the advantage of allowing adjustment of the bending moments in the deck to their optimum value.

Finally, contrary to the stress ranges in a prestressed concrete bridge, those in a cable-stayed bridge are directly dependent on the loads applied to the structure, and can attain  $100 - 150 \text{ N/mm}^2$  depending of the ratio live loads/total loads.

### 3.2 Qualities

The stay cables are an essential part of the structure, and, as such, must possess a number of qualities, among which the following are considered of paramount importance:

- high stiffness and mechanical strength,
- fatigue strength,
- durability.

#### 3.2.1 Stiffness and mechanical strength

The stiffness of a cable-stayed bridge depends, to a large extent, upon the stiffness of the stay cables which is not constant: it depends on the stress and on the cable sag, which varies as the third power of the steel stress (see section 4.6.3).

Therefore, high stress and, consequently, high strength steel must be used for stay cables.

Strand for prestressed concrete, which was first used by Freyssinet as the resisting element for bridge stay cables, is today the material most used for this application.

Fig. 8 shows the actual high strength steel ratio per square metre of deck for the three main types of bridges, deduced from a large number of structures. It can be noted that there is some scatter in the ratios, and in particular for concrete decks, the higher ratios corresponding to bridges having asymmetrical single pylons or short, heavy back spans, and the lower to decks with lightweight concrete. The span to be considered for single pylon bridges is twice the longest span, and the surface for quantity computation is the total stayed deck surface.

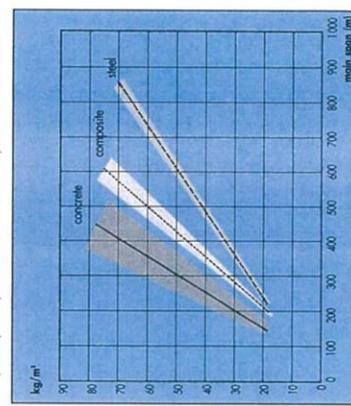


Fig. 8 - Weight of high strength steel stay cables as a function of the main span length, for various deck types.

### 3. function and qualities of a stay cable

#### 3.2.2 Fatigue strength

Repetitive loads, even small when the number of applications is great, can damage the strength capacity of the steel with premature rupture. Therefore, an analysis of all loading effects which can produce such phenomena must be taken into account.

Two complementary aspects must be considered when analysing fatigue loading:

- The first concerns the working loads to be taken into account for a large number of loading cycles.
- The second concerns the bending stresses at the anchorages, occurring when the stays are subjected to load variations or to vibration due to wind effects, which add to those resulting from pure tension.

#### 3.2.3 Fatigue and working loads

The loads to be taken into account when examining fatigue phenomena involve considerations of a probabilistic nature with regard to the frequency of the loads actually applied.

The spectra, which indicate the intensity of loading in relation to the number of loading cycles, are of fundamentally different shape when they describe a natural action, such as highway loading, as against a determinist action, such as railway loading (fig 9).

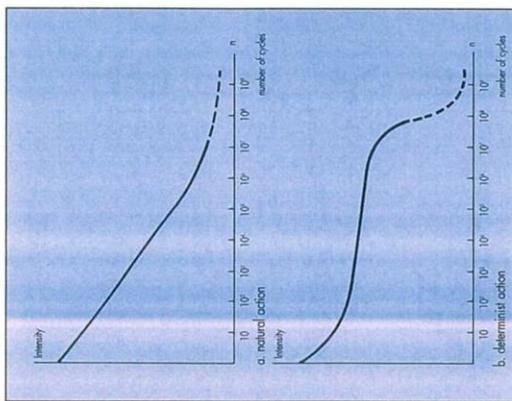


Fig. 9 - Spectra of intensity of loading in terms of the number of cycles n

Whereas the intensity of loading may be accurately evaluated for railway loading, that to be taken for road bridges is very difficult to estimate. Its inconsistent character results from the fact that the traffic on a road bridge depends on numerous

parameters which may change with time, such as the environment of the structure (e.g. industrial development, etc.).

Consequently it is practically impossible to determine, at the outset, the nature and intensity of the loads to which a road bridge will in fact be subjected.

This is one of the fundamental reasons why it is absolutely imperative that the stay cables should possess high qualities of fatigue resistance, which are verified during the preliminary acceptance tests, in order to be prepared for the effects of possible modifications in the applied loads.

The German code of practice, DIN 1075, recommends the application of the following reduction coefficients to be applied to the nominal loads:

- 1.00 for railway loading
- 0.80 for class 30 road loading
- 0.50 for class 60 road loading

In addition it demands severe acceptance requirements regarding fatigue resistance.

#### 3.2.4 Bending and tension

Stay cables are subjected to loads which induce both axial and bending stresses.

The bending stresses are negligible in the free length zone but increase towards the extremities. In point of fact, a stay cable anchorage, which, theoretically, should act as a pin joint, is in reality a fixed joint.

Consequently, a variation of tension in the stay, which results in a modification of its distortion, introduces bending stresses in the anchorage zone (fig 10).

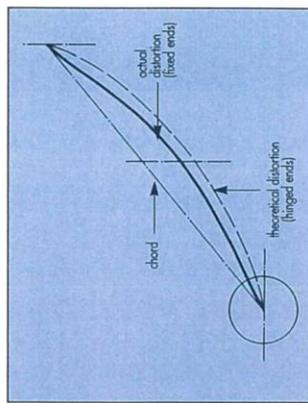


Fig. 10 - Distortion of a stay cable

Likewise, oscillations due to wind, and the deflection of the deck under applied loads, generate bending stresses which, if added to those previously mentioned and with the axial stress, call for special arrangements in order to resist fatigue effects.

By analysing the induced bending stresses, the value of the bending moment in the stay may be obtained at any point distant x from the anchorage, as a function of the angle between the tangent at the fixed end and the tangent at the distortion (fig 11).

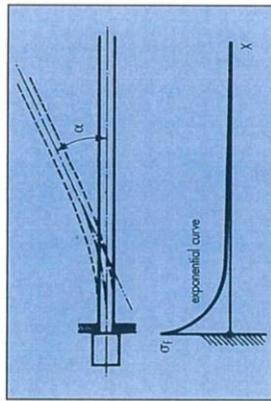


Fig. 11 - Distorted stay cable at its extremity and corresponding bending stress in terms of  $\alpha$

The bending moment at a section x is given by the equation:

$$M = \alpha \sqrt{EF} e^{kx} \quad [1] \quad \text{with } k = \sqrt{\frac{F}{EI}} \quad [2] \quad \text{and}$$

F = force in the stay cable

E = modulus of elasticity of steel

I = moment of inertia of the stay cable (see note 1 below)

The corresponding flexural stress is equal to:

$$\sigma_f = 2 \alpha \frac{d}{4i} \sqrt{EF} e^{kx} \quad [3]$$

where d is the cable diameter (see note 2, below)

i is the radius of gyration corresponding to d steel cross section of the stay.

Then, from [2]:

$$k = \sqrt{\frac{A \sigma_f}{EI}} = \frac{1}{i} \sqrt{\frac{\sigma_f}{E}} \quad [4]$$

If one assumes a uniform circular cross-section for the stay, then:

$$\sigma_f = 2 \alpha \sqrt{EF} e^{kx} \quad [5] \quad \text{since, in this case, } i = d/4.$$

From its maximum value  $\sigma_{fmax}$  at the anchorage fixed end, where  $x = 0$ ,  $\sigma_f$  decreases exponentially:

$$\sigma_f = \sigma_{fmax} e^{-kx} \quad [6]$$

with  $\sigma_{fmax} = 2 \alpha \sqrt{EF}$ , which is the reference flexural stress.

Note 1: The moment of inertia I to be considered is:

- either n times the moment of inertia of one strand which is equivalent to that of a solid bar having 0.9 x nominal strand diameter which is the case with individually protected strands;
- or, the moment of inertia of the entire group of constituent wires or strands when they are bonded together by a stiff adhesive product, e.g. cement grout.

Note 2: The diameter d to be considered is:

- either, the equivalent diameter of the strands where they are independent of each other;
- or, the equivalent diameter of the group of wires or strands, when they are bonded together, which is very close to the outer diameter of the cylinder.

For angles of deflection of the order of  $10^{-2}$  to  $10^{-1}$  radians, then:

$$\begin{aligned} E &= 200\,000 \text{ N/mm}^2 \text{ and } \sigma_f = 500 \text{ N/mm}^2 \\ &\text{(which is a usual value for permanent loads)} \\ \text{with } \alpha &= 10^{-2}; \sigma_{fmax} = 20 \text{ N/mm}^2 \\ \text{with } \alpha &= 10^{-1}; \sigma_{fmax} = 200 \text{ N/mm}^2 \end{aligned}$$

Thus the bending stress variation for restrained rotations at the anchorage may be of the same order of magnitude as the axial stress variations in case of very flexible stayed structures.

Taking this observation into account, it is recommended to take adequate steps to reduce to a minimum the bending stresses at the anchorage position.

Local reinforcement of the stay by means of a tube of appropriate thickness, bonded to the wires, is one of several solutions. Another method, both effective and economical, which is used on Freyssinet stay cables, consists of passing the cable through a guide situated at a distance l from the anchorage. The reduction in bending stress is quite considerable, once the distance is sufficiently great, as compared with the equivalent strand diameter d.

Fig. 12 gives values of the reduction coefficient  $\beta$  (or the anchorage) to apply to the maximum bending stress or reference bending stress  $\sigma_{fmax}$

$$\sigma_f = \beta \sigma_{fmax} \quad [8]$$

where:

$$\beta = \frac{kl + e^{kl} - e^{-kl}}{2} \quad [9]$$

For a ratio  $l/d = 30$ , the bending stress  $\sigma_f$  at the anchorage position becomes only 10% of the reference bending stress  $\sigma_{fmax}$ .

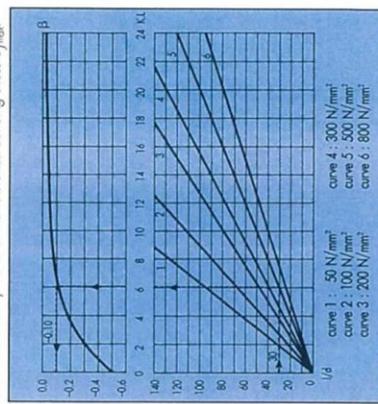


Fig. 12 - Reduction coefficient  $\beta$

# 3. function and qualities of a stay cable

Fig. 13 shows two cases of bending stress distribution:

- the first case is that of a stay composed of wires banded together (e.g. by rigid groll) where  $d$  may be taken as being equal to the external diameter 0.20 m and where a guide is placed at 2.00 m from the anchorage.
- In this case,  $l/d$  is equal to 10 and the residual value of the bending stress at anchorage is about 28 % of the reference bending stress. This value could be further reduced by local strengthening of the stay by the help of a steel banded tube.
- the second case is that of a stay composed of individually protected strands (i.e. with flexible protection), where  $d$  may be taken as that of wires having the equivalent moment of inertia  $10.0135 \text{ m}^4$  for a 15 mm diameter strand, and where the guide is placed at 0.40 m from the anchorage (internal guide). In this case,  $l/d$  is equal to 30, and the bending stress at the anchorage position is 10 % of the reference bending stress.

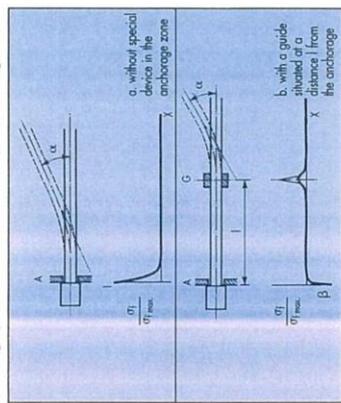


Fig. 13 - Bending stress distribution

This brief analysis shows that, if no special precautions are taken, the bending stresses are far from negligible and that it is necessary to provide devices capable of reducing them considerably. The design of the Freyssinet stay cable entirely satisfies this requirement. The bending stresses are practically non-existent within the anchorage. Only reduced bending stresses exist at the exit point of the guide, outside the anchorage gripping zone.

### 3.2.5 Durability

Stay cables, the key elements of stayed bridges, must offer the best operational guarantee, both by their nature and their design. It is for this reason that their protection against corrosion must be particularly well designed and carefully carried out. Furthermore, as it is never possible to exclude the possibility of a traffic accident, the design of the stays must allow for their rapid and easy replacement without causing any undue modification to the structure nor major disruption of traffic. The design of Freyssinet staycables is particularly oriented to satisfy these requirements and also provides for easy surveillance and maintenance.

### • Relay devices incorporated in the pylon

When the stay cables do not cross in the pylons, it is necessary to provide for transmission of the stay forces by means of a tension member which connects the anchorages. This member has to resist and transfer to the structure the horizontal and vertical components of the loads carried by the stays. This is done in various ways:

- In concrete pylons, by strapping them with post-tensioning tendons (Fig. 15a).
- In concrete or steel pylons, by providing a steel frame, consisting generally of vertical side plates, with crossmembers on which the stay anchorages bear (fig. 15.b). In concrete pylons, the shear forces need to be transferred by connectors or prestressing.

Particular attention should be paid, when relay devices are used in conjunction with adjustable anchorages in the pylons, to the clearance required to install the locking devices.

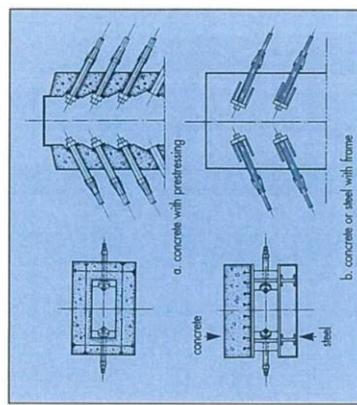


Fig. 15 - Incorporated relay devices

• **Saddles.** Although attractive from the structural point of view, saddles are, in most instances, not homogeneous with the design of other parts of the stays (Fig. 16). As already mentioned, the transfer of forces at the saddle is done through a partial length of the stay, which is not, generally, designed for that purpose. The high fatigue and static resistance requirements specified for anchorages should also be specified for the saddles. In particular, fatigue tests with ranges identical to those required for the anchorages should be specified for the saddles. The quality of a chain is the quality of its weakest link.

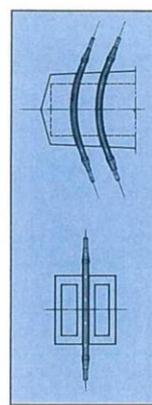


Fig. 16 - Saddle

It is possible to improve the saddle arrangements, and Freyssinet has designed special devices to overcome the drawbacks (see 4.5.4). In fact, the benefits of a saddle (owing to two anchorages, in particular), are often only apparent when considering all the parameters and effects (improvement in the transfer of forces and providing a fixed point to absorb asymmetrical loading, difficulty of installation and replacement, compliance with fatigue resistance requirements).

### 3.3.2 Deck arrangement

• **Concrete decks.** The most common arrangement of concrete decks is the transfer by steel bearing plates, with the anchorages being placed under the deck (Fig. 17). A steel tube acts as a form to provide free passage to the stay cable and its possible replacement. Particular care should be given to the design of the lower extremity to provide a drain for any possible infiltration or condensation water. Although some designs incorporate shear rings to the tube, the trend is now to transfer all the force through the plate.

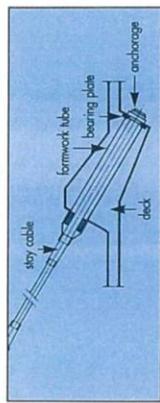


Fig. 17 - Stay cable anchorage in concrete deck

• **Steel decks.** Figure 18 shows some stay anchorage arrangements at steel decks. The attachment above the deck (fig. 18.a) facilitates the erection and inspection of anchorages, but is not well suited to adjustable anchorages. The attachment under the deck level, generally on the sides of the deck (fig. 18.b), requires access platforms.

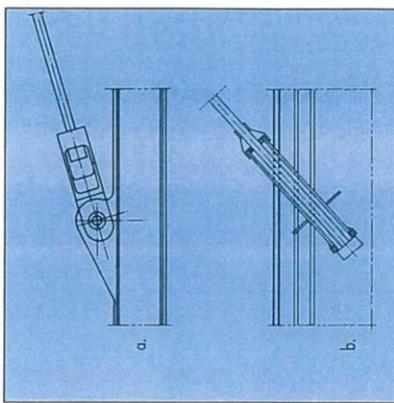


Fig. 18 - Stay cable anchorages in steel decks

### 3.3.3 Stay cable arrangement

**Typical stay cable anchorage arrangements**  
The transfer of forces from the stay cables to the structure is generally done through the extremities of the cable, where the resisting elements are firmly attached to mechanical devices called anchor heads, or, more generally, anchorages.

In concrete elements, the most common way of transmitting the stay cable force to the structure is through a bearing plate made out of structural steel, in a similar way as post-tensioning tendon forces are transmitted.

In steel structures, several solutions have been used: the anchorages may bear on steel plates which are themselves bearing on crossmembers, or directly on thick steel tubes, or still bearing (directly or threaded) on cast steel parts which are themselves fixed to the structure, or hinged.

In some cases the transfer of forces is done directly along a partial curved length of the stay cable, this being the case of the so called "saddles" in the pylon, or lower.

Detailing of the anchorage zones is of utmost importance at construction and service stages. One must bear in mind that all the operations related to the erection and finishing work of the stay cables, but also to allow for correct surveillance and inspection of the anchorage zones throughout the life of the structure. Comfortable clearances behind the anchorages will help to obtain a satisfactory condition.

Some typical examples of anchorage arrangements are included below.

### 3.3.1 Pylon arrangements

#### • Crossing of stay cables in concrete

This arrangement simplifies placing and, when necessary, replacement of a stay. The positioning of the stays is not symmetrical on either side of the pylon. This arrangement is particularly well suited for inverted Y shaped pylons (Fig. 14.a). With twin lower pylons, it is sometimes necessary to provide a simple layer on one side and a split layer on the other (Fig. 14.b). If an asymmetrical arrangement is adopted, torsion moments are induced in the pylon because two opposing stay cables cannot cross in the same plane, unless special provisions are adopted (see 4.5.4).

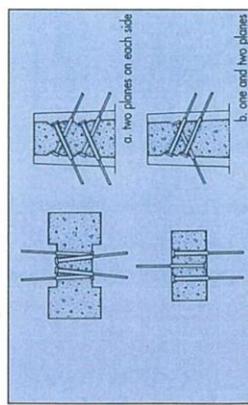


Fig. 14 - Crossing of stay cables

# 4. freyssinet stay cables

## 4.1 Description

The first large cable-stayed bridges in which stays designed by Freyssinet were used are the Pando bridge (Spain) with a 400m span steel deck and the Bionnne bridge (France) with a 320m span concrete deck. It was, at that time the largest span concrete deck cable-stayed bridge. The stays were composed of cables made of 15 mm diameter prestressing strands. The technology used at that time by Freyssinet (and some other companies) was largely inspired by prestressing technology. However, as already illustrated in section 3.1, the functioning is different, and the limits of that technology soon appeared to be insufficient to satisfy all the requirements for modern stay cables, in particular those related to fatigue resistance, corrosion protection, surveillance and replaceability.

Through intensive testing and site experience, Freyssinet has developed a modular stay cable system which can fulfil all needs for traditional and avant-garde civil engineering structures, cable roof structures, suspension bridge hangers and the like. The Freyssinet stay cable technology is based upon the total independence of each strand, and offers for the following features:

- individual anchoring;
- individual corrosion protection;
- possibility of individual erection, stressing, dismantling and replacement;

as described below:  
The usual stay service capacity ranges from 1.430 kN to 1.5144 kN at service load, although much larger stays have been supplied (24.446 kN at the Collesola tower in Barcelona, Spain). For the practical length of stays, there is no foreseen limit to date (the longest Freyssinet stay cable in the Normandie bridge, France, is 448 m long).

## 4.2 Specification of strand

The resisting element of Freyssinet stay cables is a bundle of parallel individually protected, 15 mm nominal diameter Z wire high tensile strands. The strand is hot deep galvanized before the last drawing operation, and it is then, factory sheathed with a tight high density polyethylene coating. A petroleum wax fills all the inter-wire voids and the interface strand-sheath. The strand complies with the requirements of one of the standards listed in the following table, or approved equivalent.

Strand designation	pStN 1013B3	
	Y 1770 S7-16	Y 1860 S7-16
Nominal diameter (mm)	16	EU 13B/6
Nominal tensile strength (N/mm <sup>2</sup> )	1 770	1 860
Nominal steel area (mm <sup>2</sup> )	150	150*
Nominal weight (kg/m)	1.17	1.17
Specified characteristic breaking load (kN)	265	279
Specified characteristic 0.1 % proof load (kN)	228	240
Minimum elongation at max. load (%) on L > L <sub>0</sub> (mm)	3.5	3.5

\*Available, on request, in USA.

In addition, the strand complies with the following specifications:

- Fatigue strength: 2 million cycles with  $\Delta\sigma_r = 280$  N/mm<sup>2</sup> and  $r_{max} = 0.45 \times UTS$
- Ductility (Deflected tensile strength) < 20% as per FIP
- Zinc coating 180 g/m<sup>2</sup> < weight of coating < 340 g/m<sup>2</sup>
- High density polyethylene coating mini. thickness = 1.5 mm
- Minimum quantity of wax protection = 12 g/m

Complete technical specification, data and inspection sampling and acceptance criteria are included in our technical Specification TS430697.

## 4.3 The anchorage

Each strand is individually anchored in the anchorage block, or head, which is pierced by a multiplicity of frustoconical holes, by means of three-piece jaws which have been specifically designed for high fatigue and static strength (see 4.6 mechanical performances of Freyssinet stays). There is no contact steel-steel, detrimental to fatigue resistance, other than the strand-jaw contact.

The anchorage is completed by several other components as described below, and in particular a specific Freyssinet feature which is the so called "guide/stuffing box", which plays two roles:

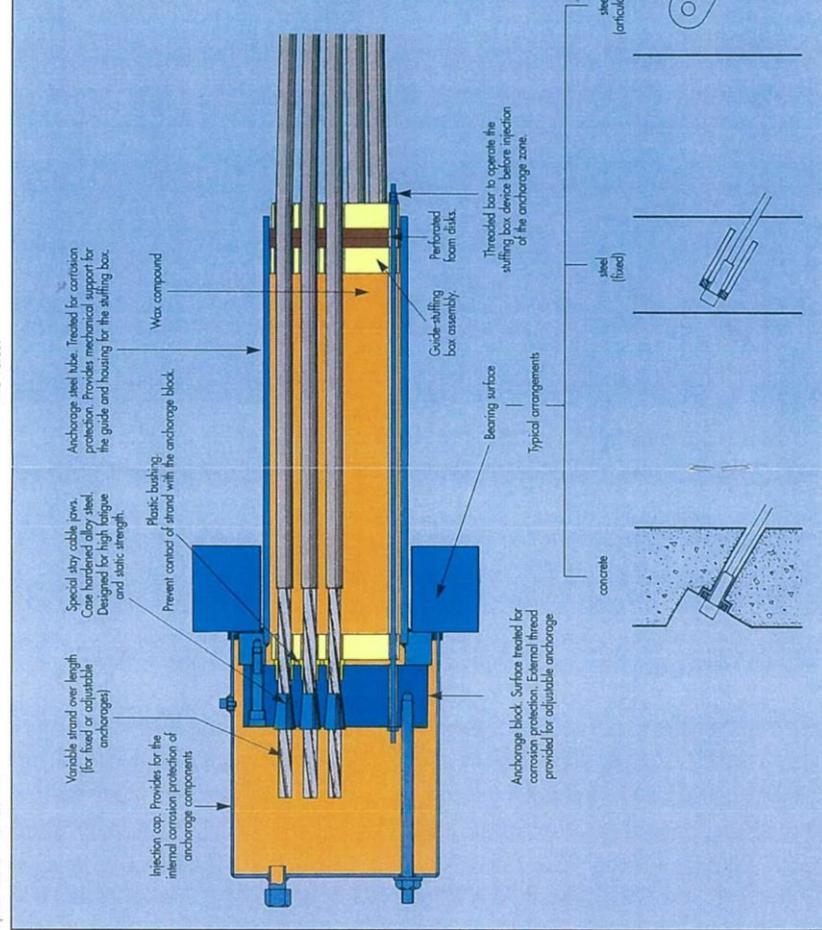


Fig. 19 - The anchorage Freyssinet

- it provides a waterproof barrier between the transition zone in which the strands are protected by a flexible compound (wax), and the zone where the Freyssinet strands are protected by the factory-applied permanent protection;
  - it is a guide placed at the proper distance from the anchorage jaws in order to eliminate the effects of local bending (refer to 3.2.4).
- A device which clamps together the strands to form a parallel bundle completes the anchorage, at a distance which is a function of the anchorage size.
- Several options and features are available to fit almost all possible structural requirements and consultant's design and specification requirements, as described in the next sections.

# 4. freyssinet stay cables

## 4.4 Anchorage types and dimensions

The anchorage described in the previous section may be used as a fixed anchorage and as a stressing anchorage (the strands being stressed one by one). This basic type of anchorage is designated as n HD 15 F, where n is the maximum number of strands that can be anchored in a given size.

However, it is sometimes specified that stay cable force adjustments should be made possible without disassembly of any part of the anchorage head (such as the injection cap, for instance, in order to reach the strand ends). In this case, what is called an adjustable anchorage should be used. This adjustable anchorage is designated as n HDE 15 R. The standard adjustment length is 100 mm.

There is a third type of anchorage, which is adjustable by means of shims placed between the anchorage block and the bearing surface (the adjustment is not as fine as with the threaded tube). The adjustment capacity of this anchorage type, called n HD 15 R, is small, and, if destressing of the stay cable is contemplated, the required amount of shims have to be provided from the installation stage.

The recommended anchorage combination for bridge stay cables is: n HD 15 F at one end, and n HDE 15 R, at the other end. Both anchorages can be placed at the pylon, or at the deck, the choice being, in general, dictated by practical considerations (space available behind the anchorages for erection and stressing, access, etc.)

Fig. 20 shows, schematically, these anchorage types, and fig. 21 the standard sizes and dimensions.

For special applications, and in particular to overcome the drawbacks encountered with saddles, Freyssinet also proposes special anchorages, such as the so called "blocked saddle", or the "crossed stays device" (see 4.5.4).

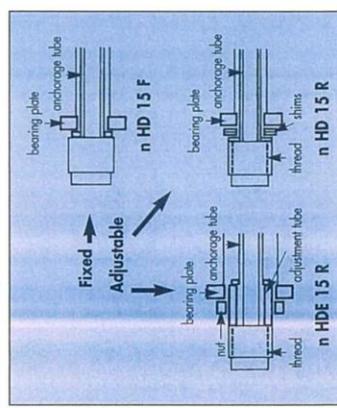
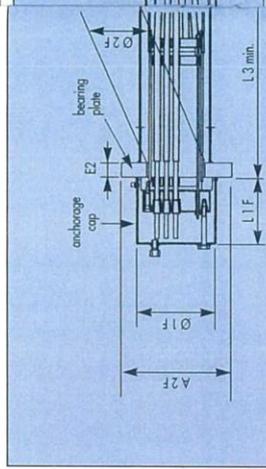


Fig. 20 - Freyssinet stay anchorage types



Fixed anchorage n HD 15 F

Adjustable anchorage n HDE 15 R

### Notes

A. The arrangement shown above is recommended for stays having:

1. No more than 0.5° of angle deviation related to the anchorage axis ;

and

2. A minimum of 2 cm clearance between the outer diameter of the stay and any part of the structure when deviated.

B. The bearing plate dimensions shown on the table correspond to anchorages placed in concrete structures.

They provide for a concrete bearing stress of approx. 30 N/mm<sup>2</sup> and have the minimum plan dimensions required for installation and stressing.

For steel structures or other bearing conditions, the minimum plan dimensions are to be respected and the thickness is to be calculated accordingly.

### Stay dimensions (mm)

Designation	N° of strands	12	19	31	37	55	61	73	91	109	127
1. anchorage	Cap O. diameter F*	200	240	280	300	360	395	425	470	500	530
	Cap O. diameter R*	200	240	280	300	360	395	425	470	500	530
	L 1 F	215	230	255	270	280	290	310	320	340	340
	Cap O. length R	490**	500**	530**	555**	580**	610**	625**	670**	690**	730**
	Nut	235	284	346	368	438	460	506	546	590	620
	Tube diameter	Ø 11 T	160	180	225	225	280	315	315	355	400
	Tube thickness	E 1 T	7.7	8.6	6.9	13.4	1.5	9.7	10.9	19.1	12.3
2. bearing	Square side dimension F	275	340	420	460	550	600	640	720	770	810
	Square side dimension R	300	350	440	470	570	610	670	750	815	850
	Central hole diameter F	Ø 2 F	131	186	221	237	285	318	342	374	386
	Central hole diameter R	Ø 2 R	192	230	270	290	350	375	405	450	480
	Thickness	E 2	50	60	70	80	90	100	110	120	130
3. other	Tube O. diameter F	Ø 3 F	168.3	219.1	244.5	273	323.9	355.6	368	419	457.2
	Tube thickness F	E 3 F	6.3	6.3	6.3	8	8.8	8.8	10	10	10
	Tube O. diameter R	Ø 3 R	219	244.5	298.5	323.9	368	406.4	445	482.6	530
	Tube thickness R	E 3 R	5.3	6.3	8	8	8.8	10	11	12.5	12.5
	Min. distance to deviator	L 3 min.	1200	1400	1750	1900	2200	2400	2500	2850	3100
	Guide tube O. diameter	Ø 3 G	193.7	203	244.5	244.5	298.5	298.5	355.6	355.6	406
	Guide tube thickness	E 3 G	5.5	5.6	6.3	6.3	8	8	8	8	8.8

\* F for fixed anchorage and R for adjustable anchorage.  
 \*\* If the foresaw adjustment length S is higher than the standard 100 mm add (S - 100).

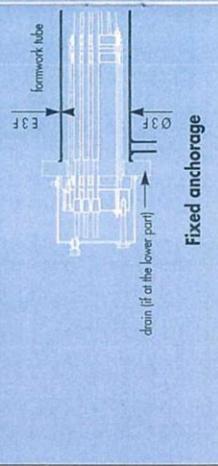


Fig. 21 - Freyssinet anchorages main dimensions

# 4. freyssinet stay cables

## 4.5 Complementary features

### 4.5.1 General

In order to provide designers with a choice of solutions to the various functional or aesthetic considerations, some complementary or alternative features are given below. They are not restrictive, but only a selection of features already used for large projects in conjunction with the present design of Freyssinet stay cables. Please contact Freyssinet for any problem or feature not dealt with in this document.

### 4.5.2 Complementary features in the free length

Due to design considerations which are, most of the time, related to aesthetic choices and sometimes to wind effects, it is possible to add a supplementary outer sheath to the bundle of Freyssinet monostrands. This sheath may consist, for instance, of thin wall polished stainless steel tube, which provides a beautiful shining finish (Wandra and Alzette bridges, see references, section 6), or of a black or coloured high density polyethylene tube (Mezcala, Glebe and other bridges). In some instances, the outer sheath may be removable, and is then fabricated in two longitudinal halves (Normandie bridge). This last solution has the advantage of providing a large wind drag coefficient reduction. Fig. 22 illustrates some of the above features.

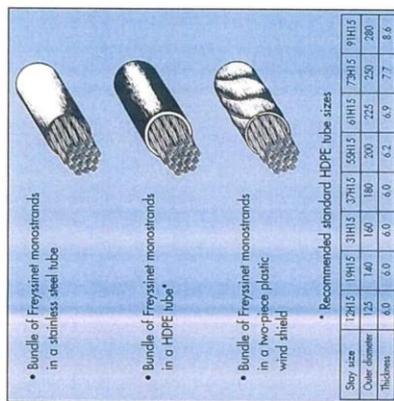


Fig. 22 - Various types of outer casings

### 4.5.3 Complementary features in the anchorage zone

It is sometimes advisable to provide an additional mechanical protection against accidental or intentional damage to the stay, (in particular to the plastic corrosion protection) in the lower zone of the stay, above the deck. This protection, called quite often anti-vandalism protection, consists generally of a steel casing placed around the stay to a height of about 3 m. The detailing of the anti-vandalism tube depends on the specific project and can hardly be standardised.

Depending on the local climatic conditions the stay vibration phenomenon may become an important factor to be taken into account. If large amplitude vibrations are likely to happen, it is wise to provide for fixing devices to allow for the possible installation of vibration dampers, should it become necessary. The dampers should preferably be of the viscous type, placed at a distance from the end of the stay for a fixed point (like the guide) sufficient to allow for some transverse movement in order to absorb energy (the sometimes called "dampers" which are simply neoprene sleeves around the stay are not, in any way, vibration dampers, but only a means of providing a fixed or semi-fixed point to minimise bending effects at the anchorage). Freyssinet has provided several types of vibration dampers in the past and may help in solving this type of problem.

### 4.5.4 Saddles

Saddles in the pylons have been used in the past, in particular with locked coil stay cables. They have also been used in connection with strand stay cables, but this type of arrangement (see 3.3.1) seems to be out of favour with designers. However, in order to respond to project tenders incorporating this arrangement, Freyssinet propose patented special devices which satisfy the following requirements:

- fatigue resistance identical to that of the stay cable;
- blocking of differential forces transmitted to the pylon;
- replaceability.

Fig. 23 illustrates two types of such devices. Additional information can be provided on request.

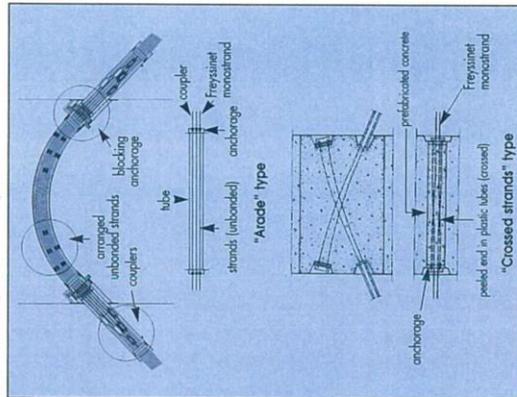


Fig. 23 - Types of special "saddles"

## 4.6 Mechanical performances

The three main parameters to be considered to assess the mechanical performances of stay cables are:

- fatigue limit;
- ultimate strength;
- stiffness.

### 4.6.1 Fatigue limit

The Freyssinet stay cable has a fatigue limit of 210 N/mm<sup>2</sup>, with an upper stress equal to 0.45 of the specified characteristic breaking load of the strand. The fatigue behaviour of the Freyssinet stay cables can be visualised in figures 24 and 25, which are the Wöhler and Smith diagrams for the constituent strand and the cable.

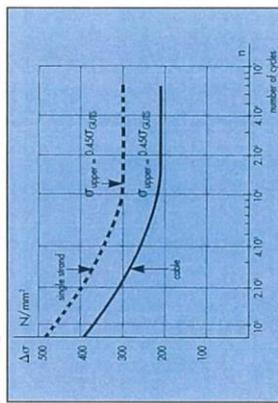


Fig. 24 - Wöhler's curves for a single strand dia. 15 mm with Freyssinet anchorages

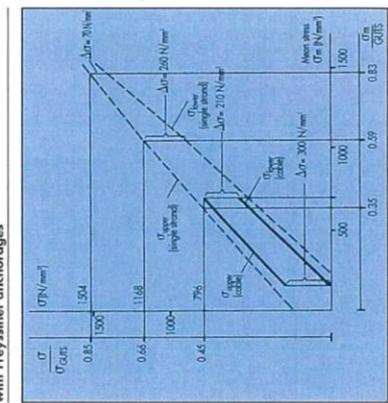


Fig. 25 - Compared Smith's diagrams for a single strand dia. 15 mm and a stay cable. Number of cycles: 2.10<sup>6</sup>

In order to facilitate dimensioning of Freyssinet stays in regard to fatigue, two curves have been drawn up, based on the Smith diagram:

- the first gives the maximum permissible stress in terms of the ratio of the extreme stress values (fig. 26):

$$\chi = \frac{\sigma_{\text{upper}}}{\sigma_{\text{lower}}}$$

- the second gives this same stress in terms of the stress variation (fig. 27):

$$\Delta\sigma = \sigma_{\text{upper}} - \sigma_{\text{lower}}$$

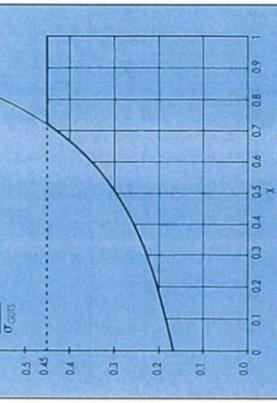


Fig. 26 - Maximum stress in terms of  $\chi = \frac{\sigma_{\text{upper}}}{\sigma_{\text{lower}}}$

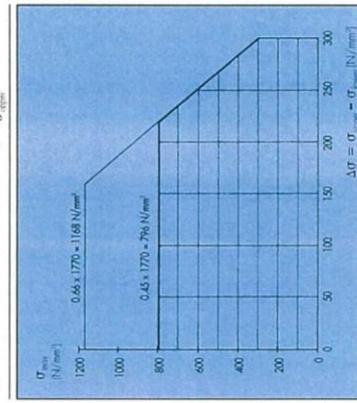


Fig. 27 - Maximum stress in terms of  $\Delta\sigma$

Both diagrams are limited to:

$$\sigma_{\text{upper}} \leq 0.45 \times \text{nominal tensile stress.}$$

They do not allow for a safety factor, this being left to the designer's appreciation.

- either by reducing the amplitude of the allowable stress;
- or by factoring the variable loads which are likely to cause fatigue problems.

# 4. freyssinet stay cables

**4.6.2 The remaining ultimate strength** is the maximum force which can be sustained by the stay cable, at the fatigue limit, without physical damage to any of its components. The Freyssinet stay cable can develop an ultimate strength equal to **95 percent** of the specified characteristic breaking load of the strand bundle (see Table 1).

**4.6.3 The stiffness** of the cable is characterised by the product  $A \times E$  of the cross-sectional area  $A$  of the cable by the effective modulus of elasticity  $E$ , and is expressed in kN (see Table 1). The effective modulus of elasticity  $E$  for Freyssinet stay cables is given in fig. 28, as a function of the horizontal projection of the stay for various levels of stress in the strand. The curves are deduced from the Ernst formula:

$$E = E_s \cdot \frac{1}{1 + \frac{\gamma}{12\sigma}} \cdot 10^{-6}$$

In which  $E$  is the effective modulus of elasticity, in N/mm<sup>2</sup>,  $E_s$  is the modulus of elasticity of the straight cable, equal to 195 000 N/mm<sup>2</sup>,  $\gamma$  is the specific weight of the cable related to the volume of steel, 87 000 N/m<sup>3</sup>,  $\sigma$  is the horizontal projection of the stay, in m and  $r$  is the axial stress in the strand, in N/mm<sup>2</sup>.

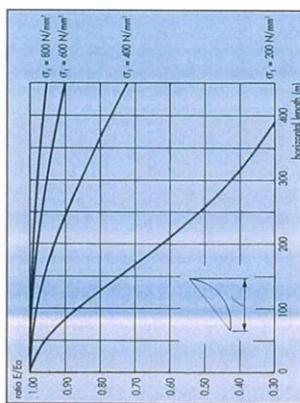


Fig. 28 - Effective modulus of elasticity  $E$  in terms of horizontal span and tensile stress in the stay cables. (Ernst's formula)

## 4.6.4 Summary of mechanical characteristics

Cable stay size	Specified 1000 Mw steel characteristic load in kN	Remaining ultimate load in kN	Stiffness $A \times E$ in kN
12 H 15	3 180	3 021	351 000
19 H 15	5 035	4 783	555 750
31 H 15	8 215	7 804	906 750
37 H 15	9 805	9 315	1 082 250
55 H 15	14 575	13 846	1 608 750
61 H 15	16 165	15 357	1 784 250
73 H 15	19 345	18 378	2 135 250
91 H 15	24 115	22 909	2 661 250
127 H 15	33 655	31 972	3 714 250

Table 1 : Mechanical characteristics of Freyssinet stay cables

**4.7 Installation**  
The Freyssinet cable stay concept, based on the independence of the strands, leads to a simple and easy installation method, which is installation of the site, strand by strand. Only light equipment is required for this operation.

The specific equipment and procedure depend to some extent on the adopted cable stay configuration and the lengths of the stays.

### 4.7.1 Isotension

Due to the flexibility of the structure, the tension of the individual strands of a stay cable varies during the stressing operation strand by strand, since the distance between the anchorages also varies during this process.

To ensure uniformity of force in all the strands of a stay at the end of the stressing operation, a patented method has been developed by Freyssinet, called **isotension**.

The principle of **isotension** is the following:

- The first strand is fixed to one of the anchorages, and its other end is threaded into the other anchorage, and then cut and stressed to a calculated force, which takes into account the foreseen deformation of the structure.
- The first strand is not yet wedged into the anchorage block, but wedged in a special single anchoring device provided with a load cell which gives a permanent reading of its force.
- The second strand is then installed in a similar manner, cut and stressed. The stressing is done, as for the first strand, with a monostrand jack equipped with a load cell identical to the one indicating the force of the first strand.
- As the second strand is stressed, the force in the first strand decreases, and the stressing operation is stopped when the readings of the two cells are identical. The second strand is then anchored in the permanent anchor block. The two strands are equally stressed.
- The third strand is then installed and stressed until its force reaches the reading of the first strand (which decreases jointly with the force of the second strand). The three strands are identically stressed.

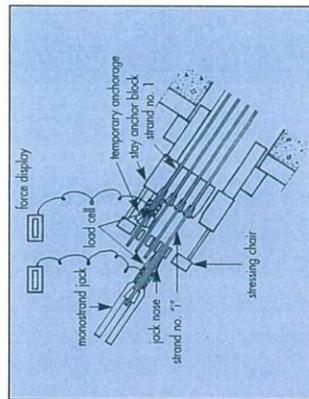


Fig. 29 - Principle of isotension

- The same operation is repeated, until the last strand of the stay. This last reading is recorded.

- The first strand is then slightly overstressed in order to allow placing of the permanent jaw in its hole. The tension of the strand is then adjusted to the last reading, and it is permanently anchored. The temporary anchor and the cell can then be removed.

The above description is, of course, very general and, in fact, the detailed procedure includes refinements to take into account all the actual parameters which may influence the operation (shortening of the stay chord during the installation, low pull in, temperature differences in the strands, etc.).

Fig. 29 and 30 show schematically the operating procedure and the principle diagram of isotension, and fig. 31 indicates the minimum clearances behind the adjustable or fixed anchorages, required for stressing by this method.

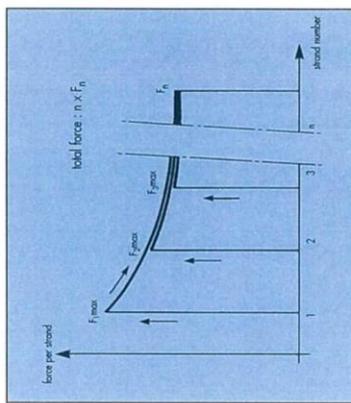


Fig. 30 - Isotension principle diagram

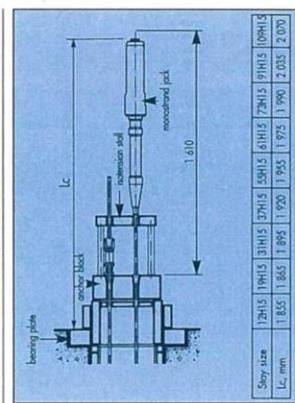


Fig. 31 - Jack clearance for isotension stressing

### 4.7.2 Tension adjustment and checking

The adjustment of the tension force in the stays can, if necessary, be carried out strand by strand in the same manner as in the initial stressing phase, by means of the monostrand jack. This operation can also be carried out, globally, by means of short-stroke jacks which Freyssinet has designed and produced for this application, for line adjustment, or when further adjustments are foreseen during the service life.

The jack bears upon the bearing plate of the adjustable anchorage, around the adjustment nut, and is screwed onto the anchorage block by means of a double threaded tube and nut (fig. 32).

The checking of the force in the stay, or "lift-off", and its subsequent adjustment, are carried out by pulling on the anchorage head until the nut loses contact with the bearing plate (force check), and then, if required, by modifying the force with the jack (over-stressing or de-stressing), and placing the nut at the required position. An opening in the base of the jack bearing ring allows for the "lift-off" stage by means of thin shims, and manoeuvre of the nut.

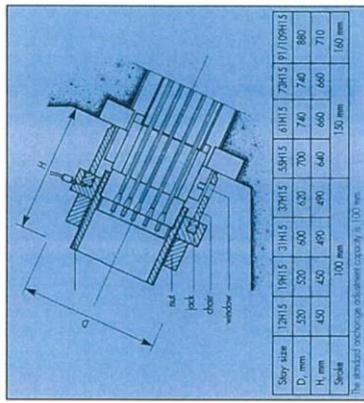


Fig. 32 - Adjustment jack bulk dimensions

### 4.7.3 Corrosion protection and finishes

The last operations to be carried out after tension adjustment and checking, if required, are the finishing of the anchorages and transition zones, and permanent corrosion protection of the exposed components.

Stay cables with individually protected strands require only simple finishing operations, such as:

- In the anchorages, cutting of the protruding strands, tightening of the stuffing box by means of the threaded rods, installation of the injection cap and filling of the anchorage with the wax.
- In the tension zone, installation of the strand deviator and guide, if required, and execution of seals and/or connections to the strand bundle or to the outer casing, depending on the options chosen for the stays.

# 5. testing

## 5.1 General

Freyssinet cable stays have undergone intensive testing on both their components and complete assemblies.

The latter have mainly been subjected to tensile (sometimes combined with bending) fatigue tests, followed by ultimate tensile strength tests. The test conditions and the acceptance criteria are, at least, those specified by the Recommendations for Stay Cable Design, Testing and Installation, published by the Post-Tensioning Institute (USA).

A summary of tests representative of the Freyssinet cable stay range is included in Table.

The tests show that the endurance of Freyssinet cable stays is 2 million cycles under a 210 N/mm<sup>2</sup> stress range, with an upper stress equal to 0.45 of the nominal tensile strength of the strand leads with lower stress ranges correspond to specific project requirements). The average efficiency is higher than 95 percent.

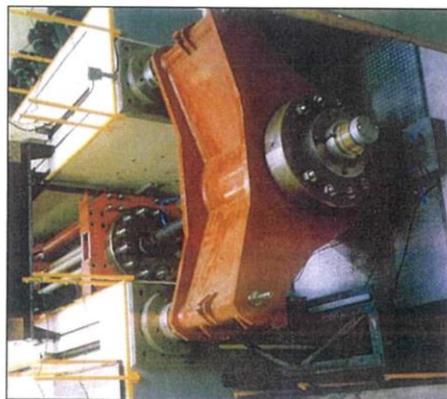


Fig. 33 - 41 strand stay cable fatigue test (LCPC, Nantes)

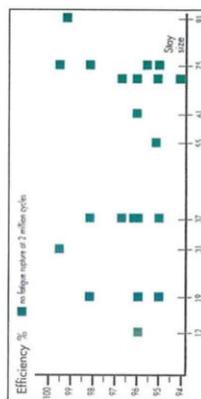
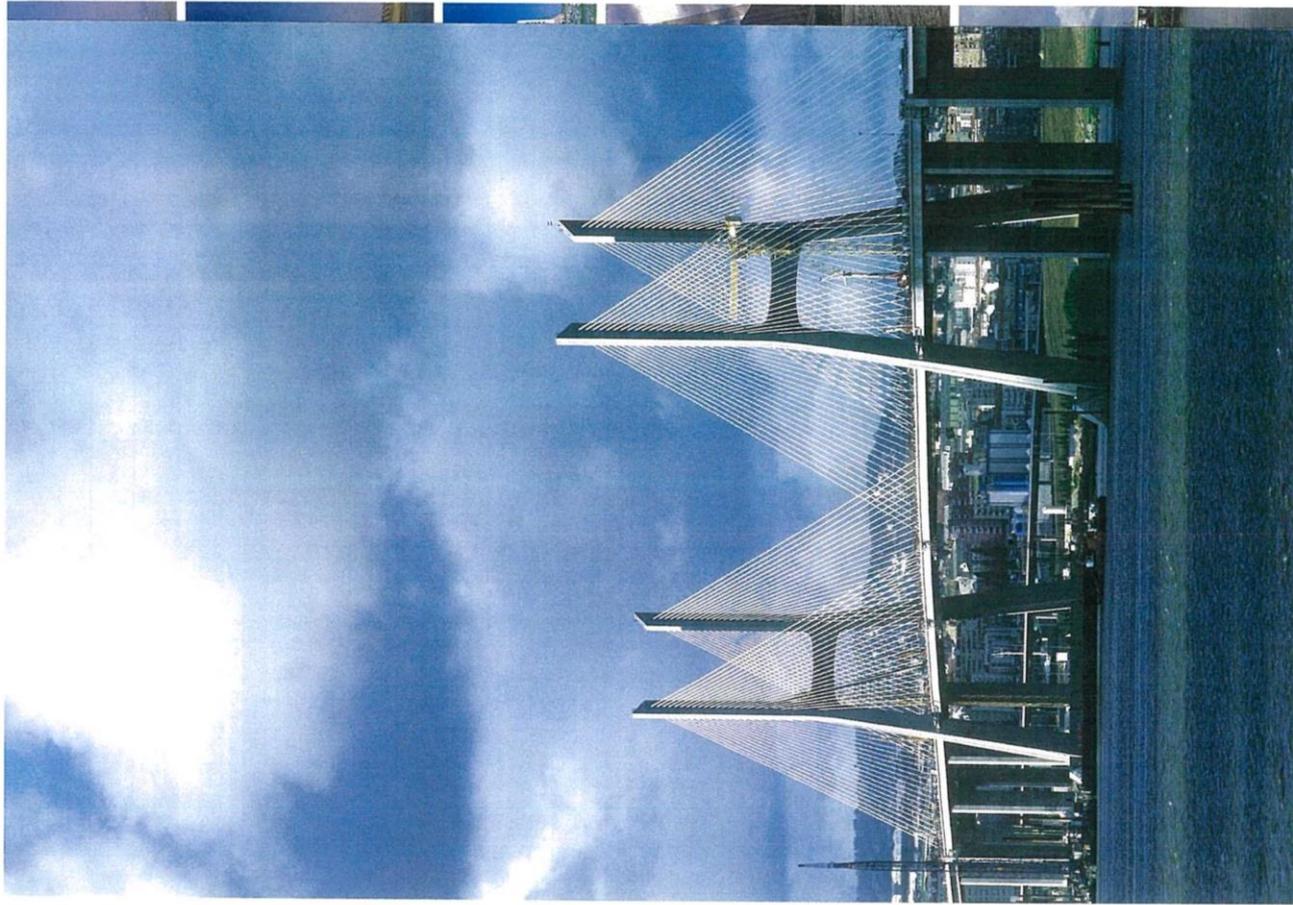


Fig. 34 - test summary diagram



Bridge name	Laboratory place, test date	Specimen: anchorage, no. of strands	Fatigue test: stress range, Result	Ultimate test: stress range, kN
Tempaco (Mexico)	EMBA Zürich (CH) 30	37 HD 15	2 million cycles	7.155
	University of Illinois (USA) 30	37 HD 15	2 million cycles	7.524 = 0.951
Harriet	University of Illinois (USA) 30	61 HD 15	2 million cycles	15.250
	March 10	61 HD 15	2 million cycles	13.850 = 0.962
	May 95	61 strands	without rupture	
Glabbe Island (Australia)	LCPC Nantes (F) 37	37 HD 15	2 million cycles	6.406
	March 93	37 HD 15	2 million cycles	6.625 = 0.967
	May 95	58 HD 15	2 million cycles	10.330
	Sept. 93	41 strands	without rupture	10.865
2nd Severn Crossing	LCPC Nantes (F) 75	75 HD 15	2 million cycles	18.850
	Feb. 04	75 HD 15	2 million cycles	19.875 = 0.948
	Feb. 04	23 strands	without rupture	
Glabbe Island (Australia)	LCPC Nantes (F) 37	37 HD 15	2 million cycles	6.385
	Apr. 04	23 strands	without rupture	6.625 = 0.964
2nd Severn Crossing	LCPC Nantes (F) 75	75 HD 15 F	1000 F cycles	19.800
	Apr. 04	75 strands	without rupture	19.875
2nd Severn Crossing (U.K.)	University of Illinois (USA) 75	75 HD 15 F	1000 F cycles	19.020
	Aug. 04	75 strands	without rupture	19.875 = 0.957
	Aug. 04	75 strands	without rupture	
2nd Severn Crossing (U.K.)	LCPC Nantes (F) 75	75 HD 15 F	2000 F cycles	19.540
	Feb. 04	75 strands	without rupture	19.875 = 0.983
2nd Severn Crossing (U.K.)	LCPC Nantes (F) 75	75 HD 15 F	2000 F cycles	19.800
	Oct. 04	31 strands	without rupture	8.180 = 0.996
2nd Severn Crossing (U.K.)	EMBA Zürich (CH) 12	12 HD 15 F	2000 F cycles	3.064
	Apr. 04	12 HD 15 F	2000 F cycles	3.180 = 0.963
Nachlaube (Germany)	LCPC Nantes (F) 19	19 HD 15 F	158 F cycles	3.560
	Oct. 05	19 strands	without rupture	3.627 = 0.981
Vanducta de (Canada)	LCPC Nantes (F) 37	37 HD 15 F	158 F cycles	8.720
	Nov. 05	37 strands	without rupture	8.928 = 0.982
2nd Severn Crossing (U.K.)	LCPC Nantes (F) 75	75 HD 15 F	177 F cycles	18.380
	Aug. 06	75 strands	without rupture	19.345 = 0.95
Aswan (Egypt)	LCPC Nantes (F) 73	73 HD 15	2 million cycles	18.600
	Oct. 06	73 strands	without rupture	19.345 = 0.961
Ches. Grandis (U.S.A.)	Ches. Grandis (U.S.A.) 19	19 HD 15 F	162 F cycles	5.049
	June 07	19 strands	without rupture	5.301 = 0.952
Osward (Denmark)	LCPC Nantes (F) 73	73 HD 15 F	2 million cycles	18.740
	Sept. 07	73 strands	without rupture	19.345 = 0.968
Osward (Sweden)	LCPC Nantes (F) 73	73 HD 15 F	2 million cycles	18.220
	Nov. 07	73 strands	without rupture	19.345 = 0.9418
Osward (Denmark)	LCPC Nantes (F) 73	73 HD 15	2 million cycles	18.483
	Nov. 07	73 strands	without rupture	19.345 = 0.954
Sevitan (Sweden)	University of Illinois (USA) 61	61 HD 15 F	158 F cycles	15.540
	Oct. 08	61 strands	without rupture	16.165 = 0.961
Charles River (U.S.A.)	University of Illinois (USA) 19	19 HD 15 F	158 F cycles	5.090
	Nov. 08	19 strands	without rupture	5.395 = 0.96
Charles River (U.S.A.)	Ches. Grandis (U.S.A.) 37	37 HD 15 F	158 F cycles	9.948
	Dec. 08	37 strands	without rupture	10.323 = 0.96
Charles River (U.S.A.)	Ches. Grandis (U.S.A.) 73	73 HD 15 F	158 F cycles	19.897
	Apr. 09	73 strands	without rupture	20.410 = 0.956
Sevitan (Sweden)	University of Illinois (USA) 61	61 HD 15 F	158 F cycles	15.540
	Dec. 08	61 strands	without rupture	16.165 = 0.96
Sevitan (Sweden)	LCPC Nantes (F) 01	01 HD 15 F	2 million cycles	24.000
	March 09	01 strands	without rupture	24.100 = 0.992

Table 2. Fatigue and subsequent ultimate strength tests on Freyssinet Stay cables.

## 6. references

The references given below relate to contracts in which Freyssinet Group member companies were involved either as a designer, a supplier or a construction specialist in stay cables.

Opening	Name	Country	Traffic type	Deck	sketch	Main span span (m)	Corrosion protection type (free length)
1977	Brienne	France	Road Bridge	Concrete		320	Cement grout in steel pipe
1977	Pasco-Kennebec	USA	Road Bridge	Concrete		299	Cement grout in HDPE pipe
1978	Kwang Fu	Taiwan	Road Bridge	Concrete		134	Cement grout in steel pipe
1978	Ronda	Spain	Road Bridge	Steel		400	Cement grout in HDPE pipe
1979	Meylan	France	Footbridge	Concrete		79	Cement grout in HDPE pipe
1980	Illied	France	Footbridge	Concrete		64	Cement grout in HDPE pipe
1980	Colton Tree Drive	Hong Kong	Footbridge	Concrete		60	Cement grout in HDPE pipe
1982	Nishikigakka	Japan	Road Bridge	Concrete		52	Cement grout in steel pipe
1983	Iuhing	USA	Road Bridge	Steel		373	Cement grout in HDPE pipe
1983	Barrios de Luna	Spain	Road Bridge	Concrete		440	Cement grout in HDPE pipe
1984	Coatzacoalcos	Mexico	Road Bridge	Concrete		288	Cement grout in steel pipe
1985	Abdijan Cathedral	Ivory Coast	-	-	-	-	Cement grout in HDPE pipe
1988	Sambre	Belgium	Rail Bridge	Composite		106	Freyssinet monostrand
1988	Tampico	Mexico	Road Bridge	Concrete/Steel		360	Galv. strand/HDPE sheath/wax
1988	Ville-sur-Haine	Belgium	Footbridge	Steel		70	Freyssinet monostrand
1988	Peruiset	France	Road Bridge	Concrete		132	Freyssinet monostrand
1988	Les Gures	France	Road Bridge	Concrete		30	Freyssinet monostrand/HDPE pipe
1989	Wandre	Belgium	Road Bridge	Concrete		168	Freyssinet monostrand/steel pipe
1989	Marbella	Spain	Road Bridge	Concrete		72	Freyssinet monostrand/steel pipe
1989	Grand Trianon	France	Footbridge	Concrete		37	Freyssinet monostrand
1990	Kamizuma	Japan	Road Bridge	Concrete		103	Cement grout in HDPE pipe
1991	Rejo Gomez	Mexico	Road Bridge	Steel		53	Freyssinet monostrand
1991	República Federal	Mexico	Road Bridge	Steel		50	Freyssinet monostrand
1991	Cristóbal Bonilla	Mexico	Road Bridge	Steel		50	Freyssinet monostrand
1991	Canal San Juan	Mexico	Road Bridge	Steel		55	Freyssinet monostrand
1991	Guadiana	Spain/Portugal	Road Bridge	Concrete		324	Freyssinet monostrand
1991	Arade	Portugal	Road Bridge	Concrete		256	Freyssinet monostrand
1991	La Blouze	France	Road Bridge	Concrete		67	Freyssinet monostrand/HDPE pipe
1991	Les Mathys	France	Road Bridge	Concrete		120	Freyssinet monostrand
1991	Labolina	Belgium	Road Bridge	Steel		60	Freyssinet monostrand/steel pipe
1991	Shiraya	Japan	Road Bridge	Concrete		124	Cement grout in FRP pipe
1991	Twin Harp	Japan	Road Bridge	Concrete		140	Cement grout in HDPE pipe
1992	Aomori	Japan	Road Bridge	Concrete		240	Cement grout in FRP pipe
1992	Bourgnone	France	Road Bridge	Concrete		152	Freyssinet monostrand
1992	Pas du Lac	France	Footbridge	Steel		68	Freyssinet monostrand
1992	Beaune Overpass	France	Road Bridge	Concrete		35	Galv. strand/HDPE sheath/wax
1992	Poplar Walkway	U.K.	Footbridge	Steel		45	Freyssinet monostrand
1992	La Barqueta	Spain	Road Bridge	Steel		168	Freyssinet monostrand/steel pipe
1992	Fargat-Alichamps	France	Telecom	Tower		-	Freyssinet monostrand
1992	Collarola	Spain	Road Bridge	Concrete		59	Cement grout in HDPE pipe
1992	Ibor	Spain	Road Bridge	Concrete/Steel		42	Freyssinet monostrand
1993	Haruta	Japan	Road Bridge	Concrete		108	Cement grout in FRP pipe
1993	Pyramon	Australia	Footbridge	Steel		42	Freyssinet monostrand
1993	Mezcala	Mexico	Road Bridge	Composite		312	Freyssinet monostrand/HDPE pipe
1993	Barranca el Zapote	Mexico	Road Bridge	Composite		176	Epoxy coated strand/HDPE pipe

\* Reinforcement of locked coil stay cables  
 \*\* Under construction

Opening	Name	Country	Traffic type	Deck	sketch	Main span span (m)	Corrosion protection type (free length)
1993	Lo Aveno	Spain	Road Bridge	Composite		105	Freyssinet monostrand
1993	Collindres	Spain	Road Bridge	Concrete		125	Freyssinet monostrand/steel pipe
1993	Alande	Spain	Road Bridge	Concrete		42	Freyssinet monostrand/steel pipe
1994	Mazau-Tajpa	Macao	Road Bridge	Concrete		112	Freyssinet monostrand
1994	Tamaracelle	Spain	Road Bridge	Concrete		162	Freyssinet monostrand/HDPE pipe
1994	Alethee	Luxembourg	Road Bridge	Composite		130	Freyssinet monostrand/steel pipe
1994	Iroise	France	Road Bridge	Concrete		400	Galv. strand/HDPE sheath/wax
1994	Rv 16 Rotherdam	Netherland	Footbridge	Steel		91	Freyssinet monostrand/HDPE pipe
1994	L'Humaité	Belgium	Road Bridge	Composite		61	Freyssinet monostrand/steel pipe
1995	Glebe Island	Australia	Road Bridge	Concrete		345	Freyssinet monostrand/HDPE pipe
1995	Zollnappel	Netherland	Road Bridge	Concrete		256	Freyssinet monostrand/HDPE pipe
1995	Normandie	France	Road Bridge	Concrete/Steel		856	Freyssinet monostrand/2 pieces wind shield
1995	Lerez	Spain	Road Bridge	Concrete		124	Freyssinet monostrand/HDPE pipe
1996	Second Severn	U.K.	Road Bridge	Steel		456	Galv. strand/HDPE sheath/wax
1996	Mei-Shyue	Taiwan	Road Bridge	Steel		210	Freyssinet monostrand
1996	Seogang Bridge	Korea	Road Bridge	Steel		150	Freyssinet monostrand/HDPE pipe
1996	Orange Overpass	France	Road Bridge	Steel		29	Freyssinet monostrand/HDPE pipe
1996	Concor East Overpass	South Africa	Road Bridge	Composite		49	Freyssinet monostrand
1997	Perera-Dasquebradas	Colombia	Road Bridge	Composite		211	Freyssinet monostrand/HDPE pipe
1997	Karkkistentalmi	Finland	Road Bridge	Steel		240	Freyssinet monostrand/HDPE pipe
1997	Dee Crossing	U.K.	Road Bridge	Concrete		194	Freyssinet monostrand/HDPE pipe/DGD
1997	No 2, 4 & 9 overpasses/A2	Portugal	Road Bridge	Concrete		64	Freyssinet monostrand
1997	Izmir-Güzelyali	Turkey	Footbridge	Composite		56	Freyssinet monostrand
1997	Two overpasses at Nubeno 2000	Italy	Road Bridge	Concrete		70	Freyssinet monostrand/HDPE pipe
1997	Oglto	Italy	Road Bridge	Composite		70	Freyssinet monostrand/HDPE pipe
1997	Tenkiji	Japan	Road Bridge	Concrete		219	Cement grout in HDPE pipe
1997	General Belgrano	Argentina	Road Bridge	Concrete		245	Freyssinet monostrand
1997	Zarate-Braco Largo	Argentina	Road & tol Bridge	Steel		330	Freyssinet monostrand
1997	West Kowloon	Hong Kong	Footbridge	Concrete		120	Freyssinet monostrand/HDPE pipe
1998	Oued Dib	Algeria	Road Bridge	Concrete		280	Freyssinet monostrand/HDPE pipe
1998	Ting Kou	Hong Kong	Road Bridge	Composite		2 x 475	Freyssinet monostrand/HDPE pipe
1998	Vasco de Gama	Portugal	Road Bridge	Composite		420	Freyssinet monostrand/HDPE pipe
1998	Santarém	Portugal	Road Bridge	Concrete		246	Freyssinet monostrand/HDPE pipe
1998	Santa Tirso	Portugal	Road Bridge	Concrete		33	Freyssinet monostrand/HDPE pipe
1998	Indoocopyly	Australia	Footbridge	Steel		180	Freyssinet monostrand/HDPE pipe
1998	Pisuegra	Spain	Road Bridge	Concrete		120	Freyssinet monostrand
1998	Archen	Spain	Road Bridge	Concrete		75	Freyssinet monostrand/HDPE pipe
1999	Sydney Airport	Australia	Road Bridge	Composite		105	Freyssinet monostrand/HDPE pipe
1999	German Technique Museum/Berlin	Germany	-	-	-	-	Freyssinet monostrand/HDPE pipe
1999	Arena Omnisports Room	Australia	-	-	-	-	Freyssinet monostrand/HDPE pipe
1999	Cardiff Arms Park Stadium	U.K.	-	-	-	-	Freyssinet monostrand/HDPE pipe/DGD
2000	Vol-Benoit	Belgium	Road Bridge	Concrete		162	Freyssinet monostrand/HDPE pipe
**	Asaon	Egypt	Road Bridge	Concrete		250	Freyssinet monostrand/HDPE pipe
**	My Thuan	Vietnam	Road Bridge	Concrete		350	Freyssinet monostrand/HDPE pipe/DGD
**	Cape Girardeau	USA	Road Bridge	Steel		330	Cement grout/HDPE sheath
**	Tarcon-Beaucarre	France	Road Bridge	Concrete		193	Freyssinet monostrand/HDPE pipe/DGD
**	Oresund	Denmark/Sweden	Road & tol Bridge	Composite		490	Freyssinet monostrand/HDPE pipe/DGD
**	Seehae	Korea	Road Bridge	Composite		470	Freyssinet monostrand/HDPE pipe



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