

第9回 鋼構造技術継承講演会

「経験豊富な先人に学ぶ次世代への継承技術」

配布資料

令和3年12月16日

土木学会 鋼構造委員会

東工大助手時代 本四疲労との出会い

本州四国連絡橋公団が富士市の建設機械化研究所に
400トンの疲労試験機を設置
高強度鋼材の溶接部や鋼橋のディテールの疲労

トラス格点部モデルの疲労試験で、理解できない結果が続出した
その一つが
試験部（格点部）からは疲労が発生せず、下弦材角継ぎ手から破壊
これが真実ならトラスの設計ができなくなる
設計部長の田島二郎博士から、一緒に研究しないかとのお願い

溶接欠陥、溶接残留応力、寸法効果、鋼材強度依存性など
まさに溶接構造の疲労研究の最先端に引きずり込まれた

学位論文：実橋での損傷事例の分析＋低サイクル疲労＋角継ぎ手の疲労



東大講師・准教授として

東大では新しい分野の研究をするように
疲労はダメ（東工大某教授からの指示）

奥村先生、西野先生から、疲労の研究を続けるようにとの指導
奥村先生が設置したアムスラー型疲労試験機（堀川先生が使用）

飯田國廣先生からの、疲労研究への強いお願い
溶接構造の疲労研究についての指導者
溶接学会、造船学会での活動

土木学会年次講演会では独立した「疲労」のセッションはなし。
最終日の最後に「疲労・継ぎ手」あるいは「溶接・疲労」の
セッションが実態



溶接構造の疲労研究とIIW

飯田先生の教え

疲労現象は疲労亀裂の発生、進展、破壊に分けて考えること
発生：局部的なひずみ
進展：Paris 則, $da/dN=C(\Delta K)^m$
破壊：Kc

疲労を説明するのは応力ではなくひずみ（塑性ひずみと弾性ひずみ）

International Institute of Welding(IIW)での活動1982-2012

研究発表及び基準類制定等につながる委員会活動
XIIIが疲労の委員会
疲労の研究者がほぼそろった場となる。

S. Maddox, P. Hargensen, P. Hobbacher, G. Marquis,.....
年次大会と中間会議、年2回顔を合わせて議論することになる

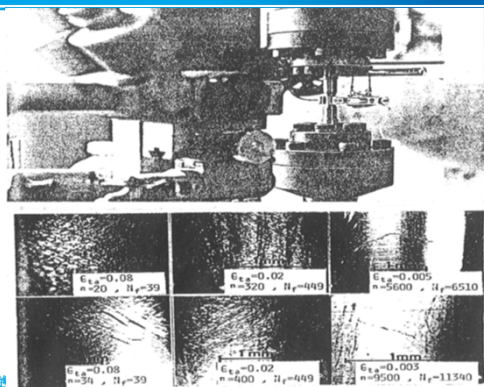


Steve Maddox と私

IIW Com.XIII, Chairman and Vice Chairman



ひずみ制御低サイクル疲労試験



本州四国連絡橋瀬戸大橋（道路鉄道併用）

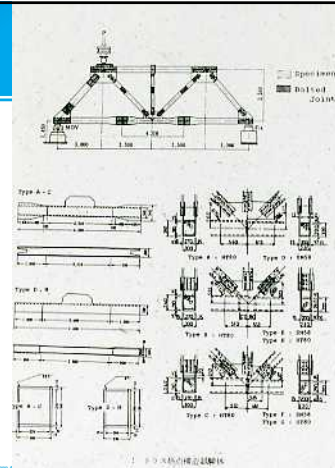


本四大型疲労試験

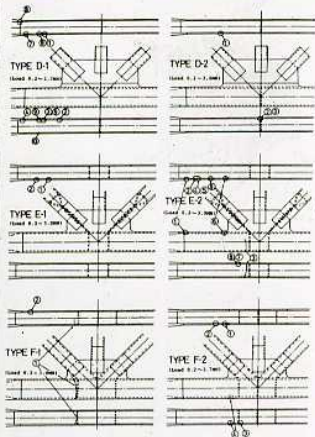
HT80、SM 58 溶接継ぎ手部の疲労強度
縦方向継ぎ手、リブ十字、ガセットなど

構造モデルの疲労
トラス格点、吊橋ハンガー取り付け部、
斜張橋ケーブルアンカー部、

トラスの格点部の疲労試験をやっている
しかし、それとは関係ない
弦材角継ぎ手から疲労亀裂が発生する
調べてほしい
田島先生、奥川さん



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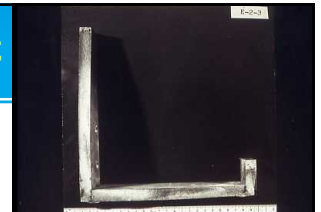


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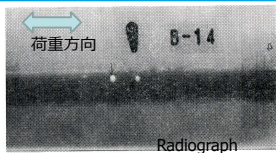
トラス弦材角継ぎ手の研究

縦ビード溶接
荷重非伝達の継ぎ手

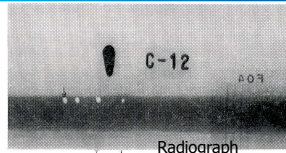
ブローホールの影響
残留応力の影響
いずれも影響は小さいとされていた



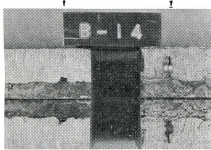
小型ブローホール内蔵継ぎ手の疲労試験 多くのブローホールから疲労亀裂が発生



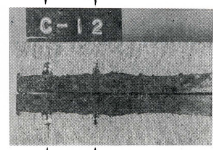
Radiograph



Radiograph



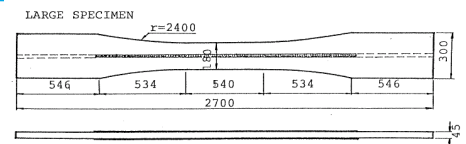
Crack



Crack

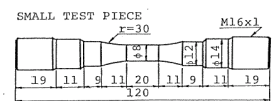
三木・西野、平林
土論1982

残留応力の影響



小型試験片は大型試験体
の溶接部を切り出したもの

大型試験体にはフルの
残留応力が存在
小型試験片では
残留応力は解放されている

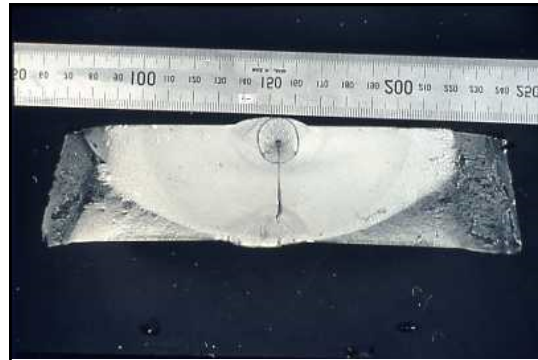


残留応力の測定からの思い付き

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ビーチマーク試験の効果

疲労亀裂の発生と進展を観察したい
応力状態が変わると破面にビーチマークが残る
それでは意図的にビーチマークを残そう
試験時間が倍増することから嫌われた
多くのことがビーチマークから明確になった
疲労亀裂の発生と進展寿命の分離
亀裂の進展挙動
亀裂の進展速度



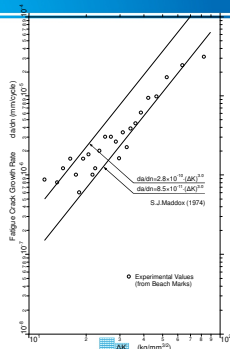
34回の荷重の半減操作」(ステップ)
32個のビーチマークが残されていた
疲労寿命のほとんどが亀裂進展寿命
これは驚き。破壊力学が使える

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ビーチマークから求まる疲労亀裂進展速度と ΔK



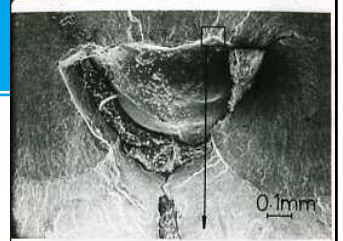
ビーチマークの間は同じ繰り返し数
ビーチマークの間隔から進展速度がわかる



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疲労亀裂の起点は大きさ0.7mm程度の
ブローホールの壁

き裂発生点(ビーチマークの原点)には
ステップ上の痕跡

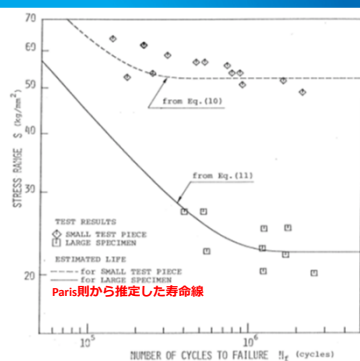


残留応力の影響

疲労強度の差が残留応力
大型試験体、小型試験片
とも縦ビード溶接ルートの
微小なブローホールから
疲労亀裂が発生

寿命のほとんどが亀裂進展
に使われる

破壊力学適用で寿命推定が可能
Paris則: $da/dN = C(\Delta K)^m$



三木・田島、土論1982

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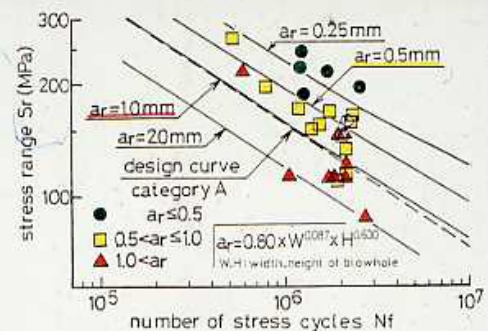
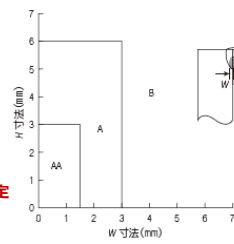


Fig. The effect of blowhole size on the fatigue strength of corner joint

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部材の疲労に対する厳しさと溶接要求品質とのリンク

角継ぎ手部のルート部に発生する
ブローホールを対象として
疲労許容応力度と受け入れ限界欠陥の規定
世界初のFitness for Purpose design と評価

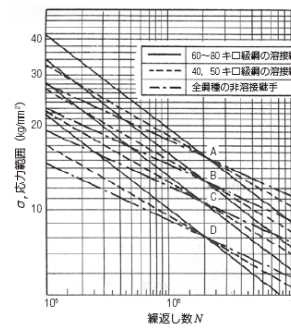


	S _{TS}	受け入れ限界欠陥の寸法	UTの検査率
最も厳しい: AA	0.7 < S _{TS} /S _u	W ≤ 1.5, H ≤ 3.0	全体
厳しい: A	0.5 ≤ S _{TS} /S _u ≤ 0.7	W ≤ 3.0, H ≤ 6.0	全体
普通: B	S _{TS} ≤ 0.5	H	20%

角溶接部に対する疲労の厳しさとそれに対応した受け入れ限界欠陥

S_u: 応力集中に対する応力
S_{TS}: 応力集中と応力集中に対する応力
S_{TS}が0.7より大きい(応力集中のうち応力集中の割合が70%と高い)
場合はAA部材と分類され、その際の受け入れ限界欠陥はW ≤ 1.5mm,
H ≤ 3.0mmとなる

図-8.9 トラス試験体に発生した疲労亀裂の分布



	鋼種	等級分類	m	Constant
溶接	40 50 千口級	A	4	1.10×10^{11}
		B		5.20×10^{10}
		C		2.43×10^{10}
		D		8.19×10^9
継手	60 80 千口級	A	3	7.16×10^9
		B		4.10×10^9
		C		2.31×10^9
		D		1.02×10^9
非溶接継手	全鋼種	A	5	1.68×10^{12}
		B		6.61×10^{11}
		C		2.55×10^{11}
		D		6.55×10^{10}
$(\sigma_r)^m \cdot N = \text{Constant} \quad \sigma_r: \text{kg/mm}^2$				

(σ_r)^m · N = Constant σ_r: kg/mm²

疲労許容応力度をS-N表示
強度逆依存

Nは1000万回: 100年間での新幹線列車と総貨物列車通過トン数から決定

本四疲労の設計基準類への反映

本四疲労設計指針 S-N線ベースの疲労設計
材料非依存、逆依存

鉄道橋の疲労設計 本四疲労の反映
S-N線ベース
継ぎ手等級の見直し

鋼構造協会疲労設計指針の全面改定

道路橋示方書への疲労設計の導入

ASSHTO Spec., IIW疲労設計指針 にも影響

本四疲労は1990年ころまで。様々な構造要素の疲労性能の改善に貢献。
疲労試験設備は今の現役として働いている。

溶接構造の疲労研究の成果

機械部品(機械加工品)とは全く異なる世界

Lehighの研究, TWIを中心としたヨーロッパの研究, 本四の研究
(Fisher) (Maddox)
ほぼ同じ時期に同じような結果、

- ・鋼材強度非依存、逆依存: Lehigh, 本四
- ・溶接残留応力の効果: 疲労強度に支配的な影響、応力比非依存
- ・溶接欠陥の影響: ブローホールのような丸い形状でも強く影響
面状欠陥は亀裂と同じような効果
- ・N_cとN_p: N_cは極めて小さい。m=3、ただし、疲労限界は存在する
- ・寸法効果と板厚効果:
- ・変動応力: 打ち切り限界、長寿命域でのm

破壊力学の役割: きわめて有効なコンセプト
ただし、説明できることとできないことがある

Lehigh大学



Lehigh University, Fritz Engineering Laboratory



日本人PhDは約20名(土木、建築、造船、など。鋼構造研究の中心)
西野文雄先生、福本先生、藤田譲先生、川合忠彦先生、・・・
柱や桁の耐力研究、疲労は少数派

Lehigh での研究

J.W. Fisher 教授との出会い

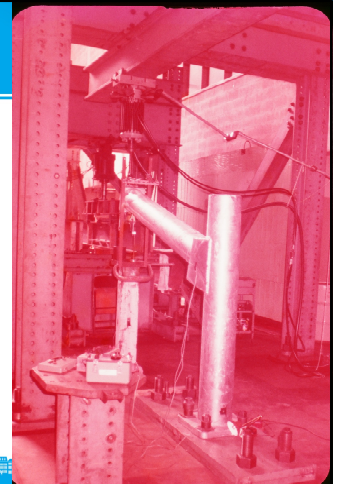
1982年 最初に担当したProject: Light Poleの柱と横梁の接合部の疲労、英語が通じない。Work orderが書けない。
Beach Mark Testによりき裂の発生と進展とを分離して示した。
Prof. Fisherとは週1回1時間の面談時間
朝の疲労試験巡回時7:30とCoffee time10:30での雑談的打ち合わせ
橋の疲労損傷の調査と応力測定に同行。徹底した現場主義を学ぶ。

1984年 文部省在外研究員
余裕のある研究生活
沢山の現場を経験



1980年代
照明柱の疲労が問題となった
原因は風による振動
横梁と柱のconnection,
Base-plate Connection

California州道路局からの委託



Engineering Structures
Volume 5, Issue 2, April 1983, Pages 90-96

Fatigue strength of steel pipe-base plate connections ☆

John W. Chihler, Chiozshi Maki, Roger G. Slutter, Dennis R. Mertz, William Frank

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[https://doi.org/10.1016/0141-0296\(83\)90022-6](https://doi.org/10.1016/0141-0296(83)90022-6)

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Abstract

The comparative fatigue resistance of two types of steel light poles is investigated, both experimentally and analytically. The fatigue tests were performed on test specimens designed and proportioned by the California Department of Transportation. The constant amplitude fatigue behaviour was obtained at several levels of stress range. Exposed fatigue crack surfaces were studied to ascertain the nature of the crack initiation and propagation. Theoretical fatigue life estimates were also made using an existing fracture mechanics model, thought to simulate the geometric condition represented by the welded pipe-base plate connection. The fatigue resistance of the two series of specimens was much lower than originally anticipated. The fatigue resistance was found to be comparable to either category E or E', depending on the weld contact angle.

Previous

Next

当時はAmerica in Ruinsの真ただ中

Fisher 教授は疲労損傷橋梁の調査の中心



道路1982-11、三木

写真-2 重量制限された橋 (ペンシルベニア州)

Cable breaks close Brooklyn Bridge walkway

NEW YORK (AP) — Pedestrians and bicyclists were forced away from the Brooklyn Bridge Tuesday morning after two cables snapped, causing the bridge to close for several hours.

The cables, which are used to support the bridge's deck, were damaged by the falling cables, said a spokesman for the New York State Thruway Authority.

The bridge, which is one of the most famous in the world, was closed for several hours after the cables broke.

The bridge is a landmark in New York City, and the closure caused many problems for commuters.

The bridge cables, which are made of steel, were damaged by the falling cables, said a spokesman for the New York State Thruway Authority.

The bridge is a landmark in New York City, and the closure caused many problems for commuters.

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Newman checks walkway holes after two cables broke.

Williamsburg Bridge Open on December 19, 1903 (110 years)

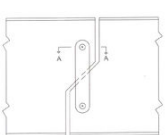






Mianus Bridge (米国)
 1983.6.28

- 伸縮装置からの漏水により、ゲルバー部の吊材のピンが腐食、疲労で破断し、落橋した
- コネチカット州の技術者が少なく、十分な点検を行えなかったことも原因と言われている
- 3名死亡

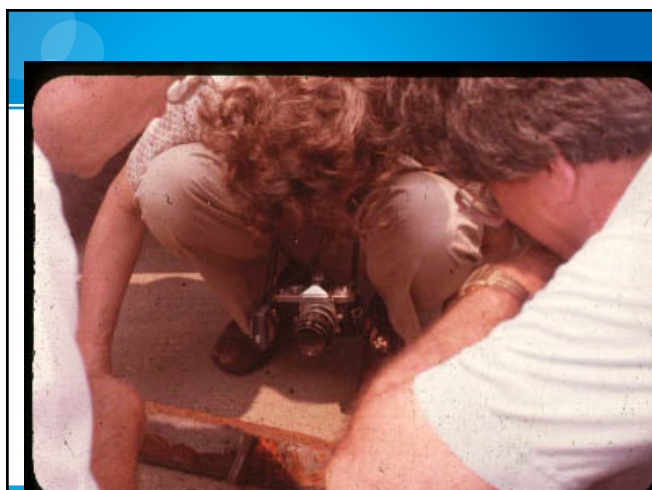

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疲労き裂とそれを起点とした脆性破壊



40



Prof. J.W.Fisherとの継続的關係、夏休みはLehigh、その後もしばしば滞在

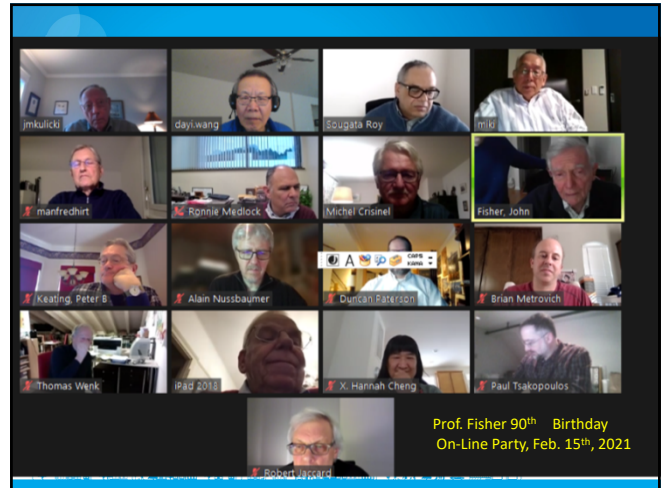
I-35W over the Mississippi River
ミネアポリス, USA、Aug.1, 2007. 6:00p.m.



Open:1967
8 lanes
140000-
200000/day



東京都市大学



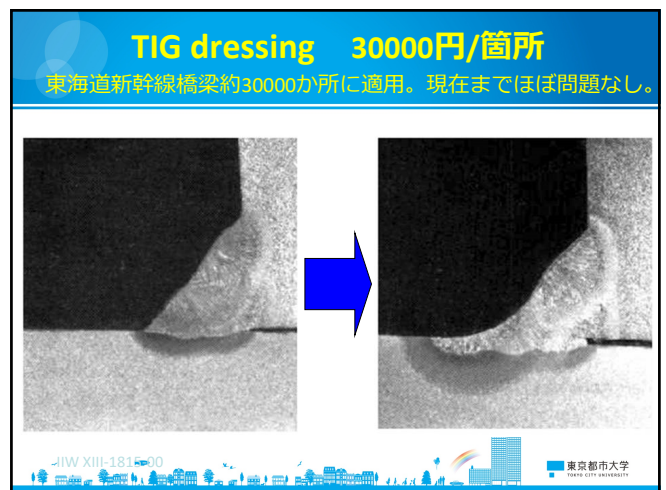
鉄道橋の疲労対策

西村俊夫教授、田島二郎教授、阿部英彦教授の流れ
(国鉄構造物設計事務所)

在来線 (JR)
当初より疲労設計
適切なメンテナンス
列車重量のコントロール
高経年、100年に近づきつつある

東海道新幹線
初めての溶接構造
疲労設計 (70年を想定) 一列車本数の増加、
当時の疲労設計と現在の疲労設計の差
開通後10年くらいから疲労亀裂が見つかり始める。
2次的な応力や振動を原因とする疲労
構造物設計事務所と一緒に原因究明と補修補強
* 疲労設計対象外の個所での損傷、2次応力疲労、振動疲労
* 予防保全としてのTIG適用 1990—
* 疲労許容応力度の見直しに伴う健全度評価
* 大規模修繕プログラム、100年を目指す

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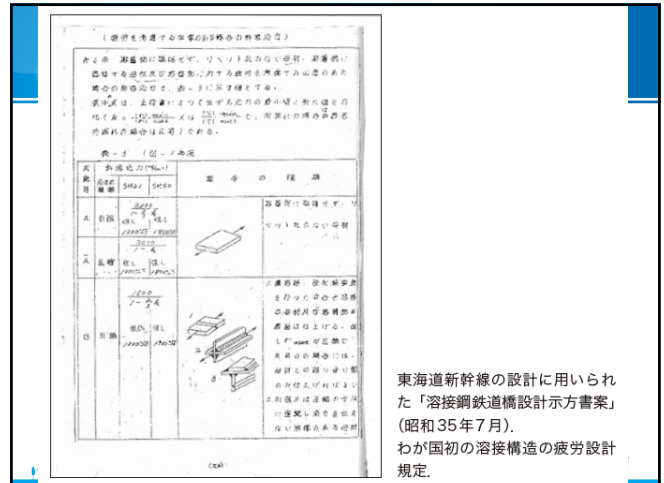


設計疲労許容応力度の見直し

本四疲労による新しい知見の反映

(本四の疲労設計は鉄道橋との併用部材のみ)

- 200万回は疲労限界ではない
- 応力比異存はなし 応力範囲
- 多くの継ぎ手で非安全
- 鋼材依存性なし



鉄道橋設計標準。1960年と現在

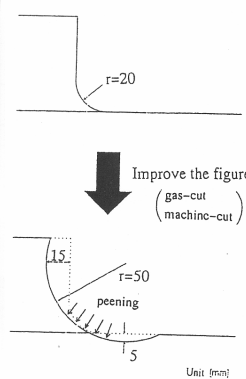
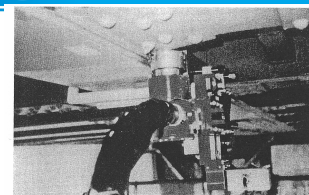
Joint Type			Joint Type		
Sleeper pad			Out-plane gusset (t=20mm)		
Code	Class	Fatigue strength at 2 million cycles (MPa)	Code	Class	Fatigue strength at 2 million cycles (MPa)
Old code '60	D	120	Old code '60	D	100
Current code 92	F	65	Current code 92	F	65

Joint Type			Joint Type		
Longitudinal welding			Cruciform weld		
Code	Class	Fatigue strength at 2 million cycles (MPa)	Code	Class	Fatigue strength at 2 million cycles (MPa)
Old code '60	B	150	Old code '60	D	100
Current code 92	C	125	Current code 92	E	80

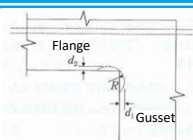
Joint Type		
In-plane gusset (t=20mm)		
Code	Class	Fatigue strength at 2 million cycles (MPa)
Old code '60	C	126
Current code 92	F	65

疲労強度改善の例:

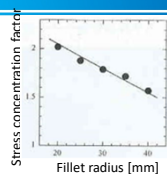
面外ガセット止端部の切削加工(応力集中を下げる)



実橋で実現可能な切り欠き加工と応力集中



(a) Cut out of gusset end corner



(b) Stress concentration and radius of fillet end R

Modification of the shape of flange gusset end to reduce stress concentration

設計では $R = 20\text{mm}$

実施工では実現できていなかったことが問題

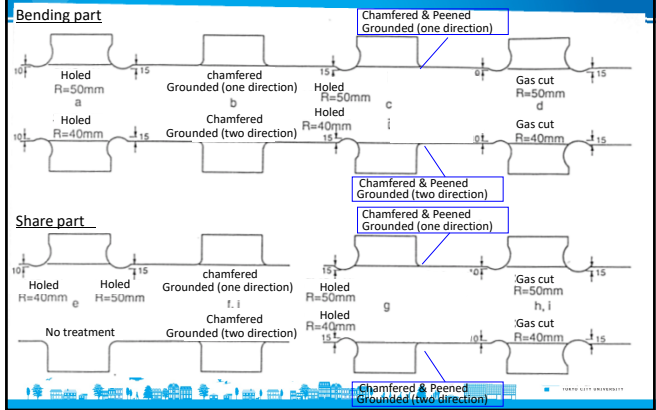
機械加工が必要



Fatigue Strength Improvement -Grinding-

Flange Gusset Specimen

The diagram illustrates the geometry and dimensions of a Flange Gusset Specimen. The side view (top) shows a specimen with a total length of 2000 mm, divided into sections A (1800 mm), B (200 mm), and C (200 mm). The top flange has a thickness of 10 mm and a width of 450 mm. The bottom flange has a thickness of 10 mm and a width of 450 mm. The gusset plate is 10 mm thick and 450 mm wide. The specimen is labeled "1-FLG PL 250X18X18 200". The end view (bottom) shows a specimen with a total length of 2000 mm, divided into sections A (1800 mm), B (200 mm), and C (200 mm). The top flange has a thickness of 10 mm and a width of 450 mm. The bottom flange has a thickness of 10 mm and a width of 450 mm. The gusset plate is 10 mm thick and 450 mm wide. The specimen is labeled "1-FLG PL 250X18X18 200". The grinding details (detail-a) show a 45° grinding angle and a 10 mm grinding depth. The grinding is performed on the top and bottom surfaces of the gusset plate, with a grinding width of 100 mm and a grinding depth of 10 mm. The grinding is performed on the top and bottom surfaces of the gusset plate, with a grinding width of 100 mm and a grinding depth of 10 mm.



Fatigue Test Result

Upper Flange

Stress range (MPa)

Number of cycles

Legend:

- r=20mm
- △ r=40mm
- ◇ r=50mm
- + r=20mm (peening)
- r=40mm (peening)
- ⊗ r=50mm (peening)

JSSC E

JSSC F

Lower Flange

Stress range (MPa)

Number of cycles

Legend:

- r=20mm
- △ r=40mm
- ◇ r=50mm
- + r=20mm (peening)
- r=40mm (peening)
- ⊗ r=50mm (peening)

JSSC E

JSSC F

Cracked from spatter

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“100年”新幹線へ

東京ー新大阪間(計515・4キロ)のうち、
鉄橋233カ所(計22キロ)
トンネル66カ所(計68キロ)、
高架橋(計98キロ)など
計約240キロで、
工期は平成34年度末までの10年間、
費用は計7300億円を見込む。
橋の架け替えなど大がかりな工事をせず、
補強などの改修だけで強度を保つ、
運行やダイヤへの影響は避けられる見込み。



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疲労強度の改善

舟形対策工法(変位の拘束)

図1 対策前

図2 対策後

舟形部材

[illegible]

道路橋の疲労損傷と疲労設計

1980：
・ランガータイプのアーチ橋垂直補剛材取り付け部：風による疲労
・東名高速道路の対傾構取り付け垂直補剛材
現場では困った、困った。本社では疲労亀裂なんて発生するはずがない。

1885：
・垂直補剛材上端部の亀裂、一般化
・ソールプレート取り付け部の疲労
・様々な疲労亀裂、下フランジの板継ぎ溶接にも発生。
土木学会に疲労に関する小委員会設置

1990：道路協会に疲労損傷調査のWG 設置、1995年までの事例のとりまとめ
道路橋示方書に疲労設計導入の準備

2000：首都高速道路疲労対策室設置；鋼製橋脚隅角部の疲労問題
全国に波及、対策

2002：橋梁委員会で、疲労設計を示方書へ入れること時期尚早、見送り→別冊の指針

2012：部分的に導入、2017：全面的に導入、
設計で想定する寿命100年もここで導入（提案は2002年）

実橋の疲労損傷の原因調査と対策は研究ネタの宝庫

損傷の原因調査、補修補強対策などから派生した研究

疲労破壊のモードの構造的同定

Load Induced Fatigue（1次応力に起因する疲労）

Displacement or Distortion Induced Fatigue（変位誘起疲労）

・自動車荷重の実態把握

・Weigh in Motion

・3次元的な変位の把握

・応力測定法

・実応力比

・

・疲労亀裂検知非破壊検査

・疲労亀裂の寸法測定非破壊検査

・疲労亀裂発生伝播のセンシング/
モニタリング

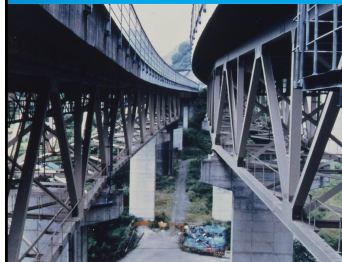
・疲労強度改善法：TIG, Peening, UIT,

・構造改善法

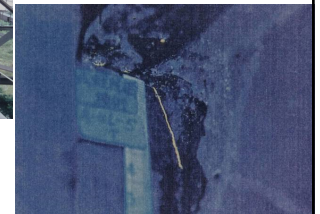
管理事務所。見慣れない亀裂が見つかった、どうしよう。→ 東工大へ
本社技術部。わが社の道路橋に疲労など発生するはずはない。



トラス、主構上弦材と横トラスとの接合部



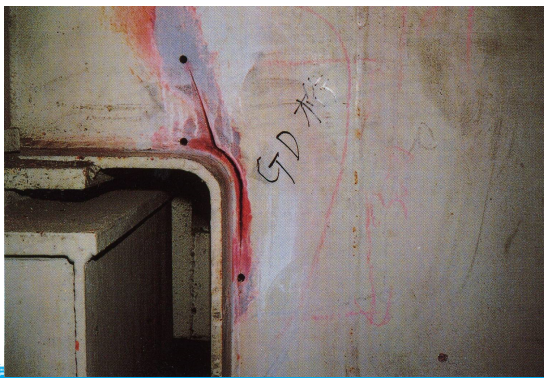
設計では単純支持
フランジ間をつなげたディテールの構もあり
そのディテールでは亀裂の発生はなし



桁端切り欠き部の疲労

小さいR：形状からくる応力集中

製作管理：フランジ・ウェブの溶接、ギャップ管理、溶接変形



支承部ソールプレート取り付け溶接前縁の亀裂

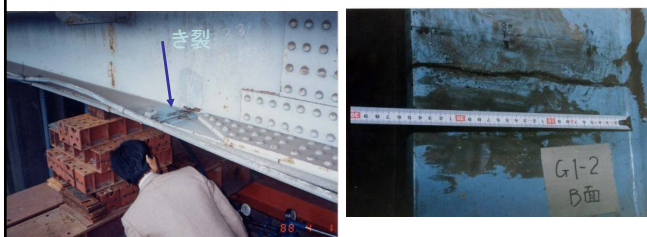
曲げ応力（公称応力）はゼロ

支承機能のロス（スライディング、回転）

溶接変形



下フランジ板継溶接部の破断



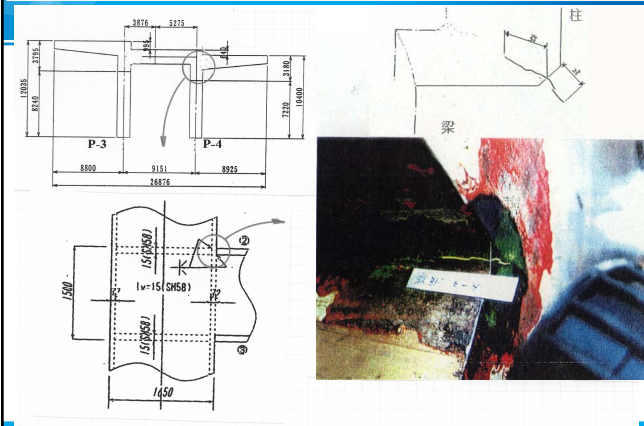
首都高速道路鋼製橋脚隅角部の疲労対策

2001～2005年、
組織（疲労対策室）を作り対応
2000基の橋脚のうちの700基に何らかの異常
300基については補強対策。

他機関の橋脚も同様
設計と製作の問題



設計では完全溶け込み溶接：



溶接部の疲労強度改善研究

**満足な結果が出ていない、
やり残し感。**

橋梁への適用性を**実大モデル**で確認
残留応力、実働応力の評価
改善効果、経済性など

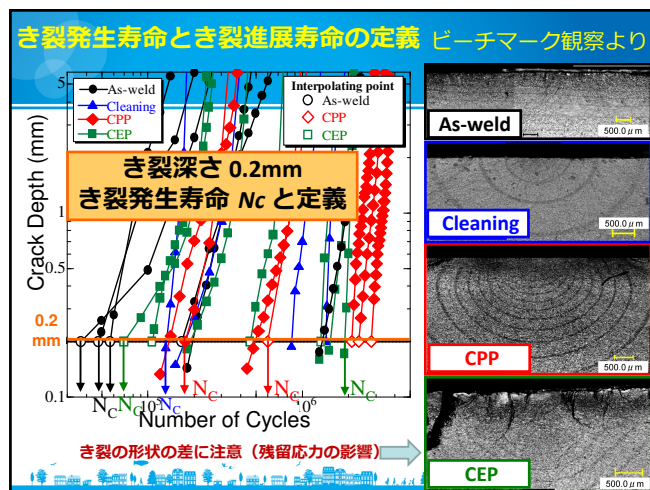
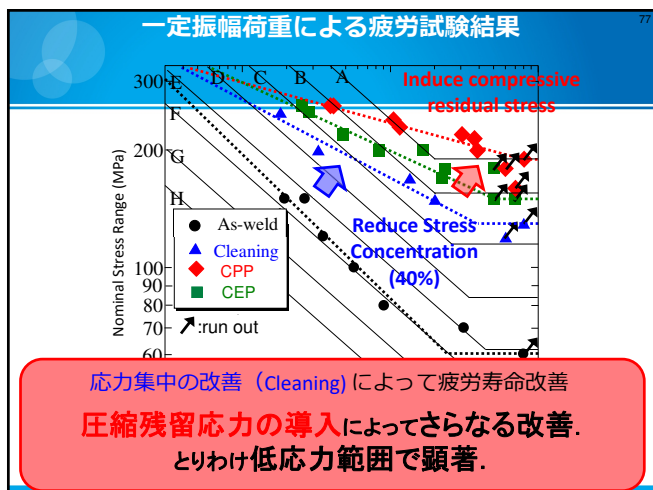
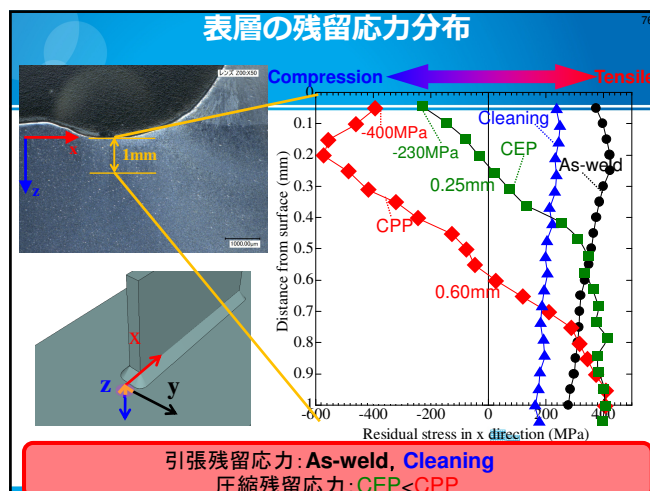
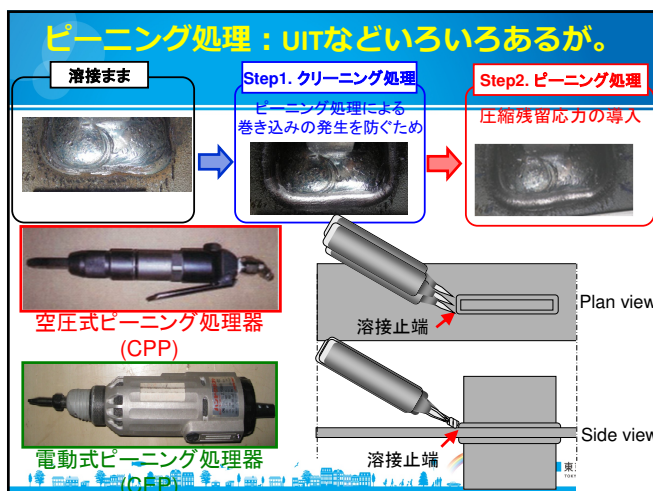
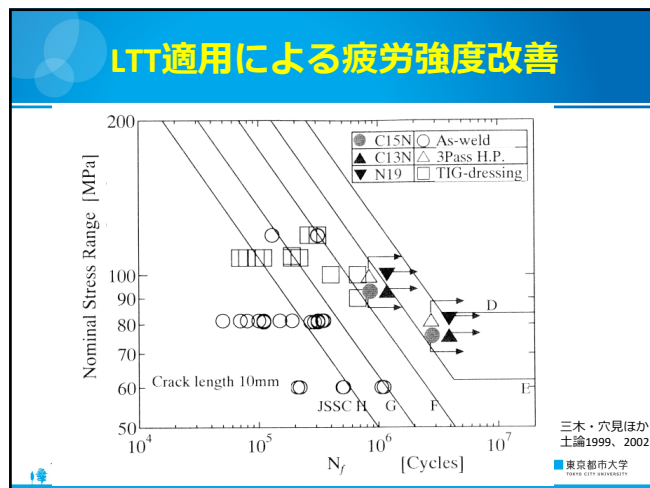
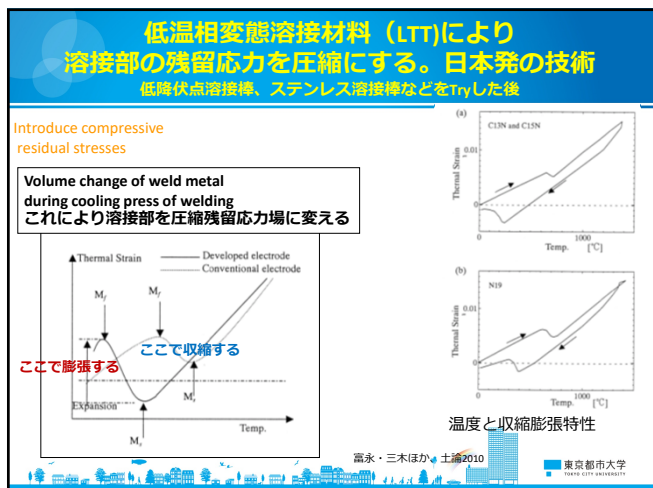
低温相変態溶接棒：日本発

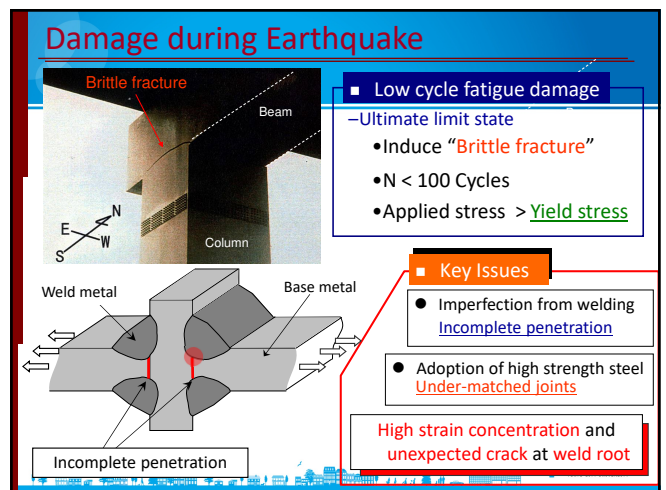
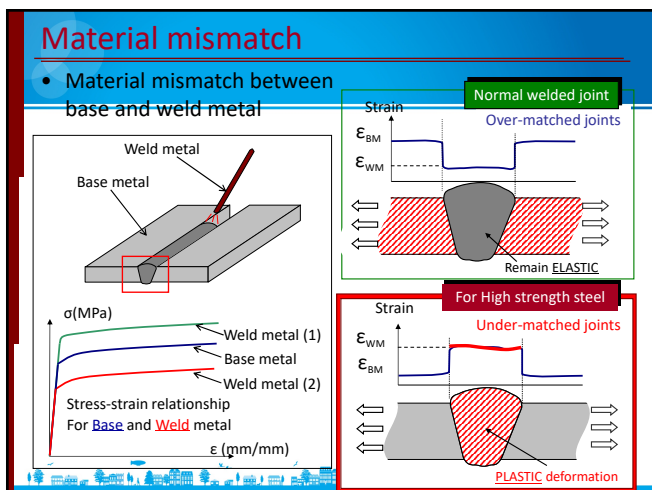
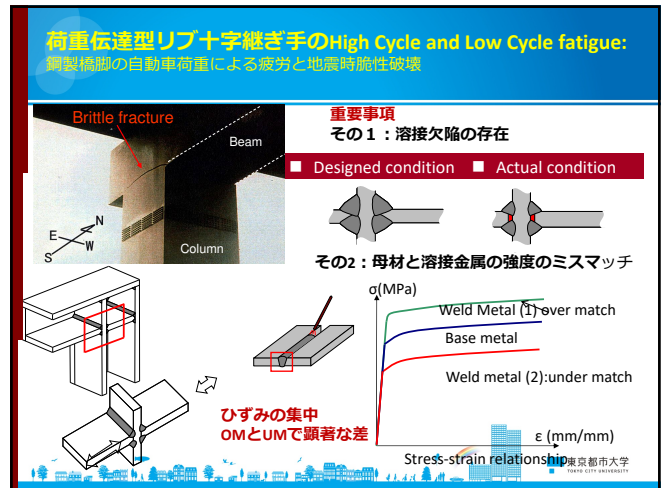
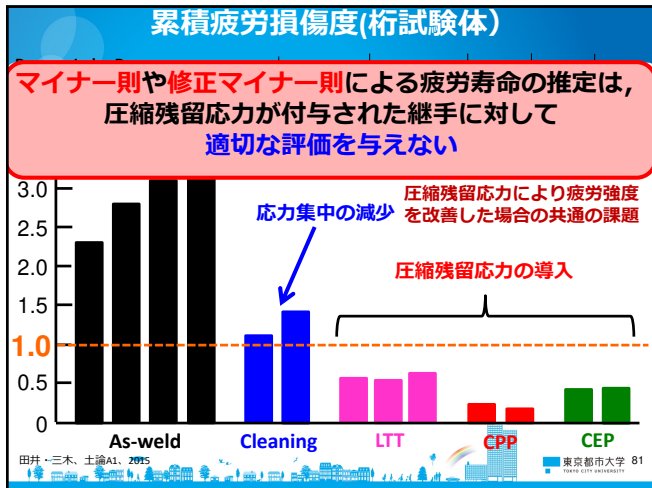
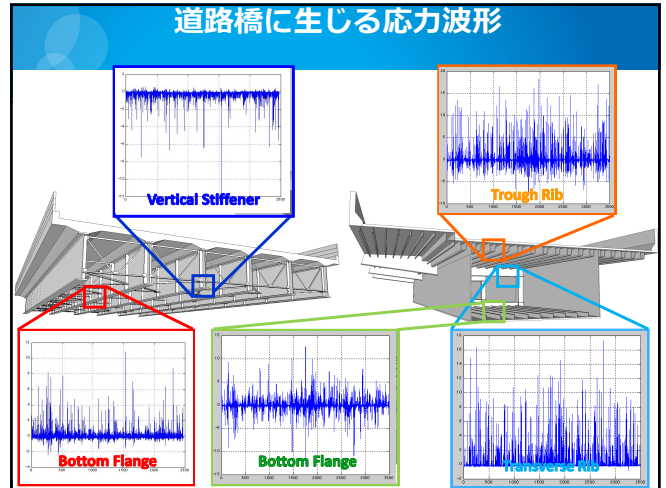
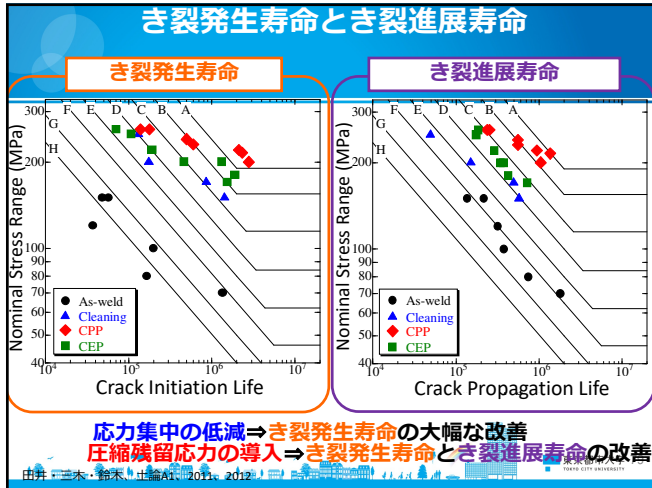
Peening処理：昔からある技術、最近はUIT等

過去、TIG, Grinding, 軟質溶接棒などの研究

実橋ではルート亀裂が多い。これは難しい。

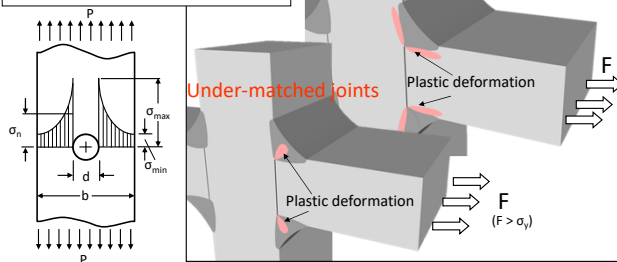






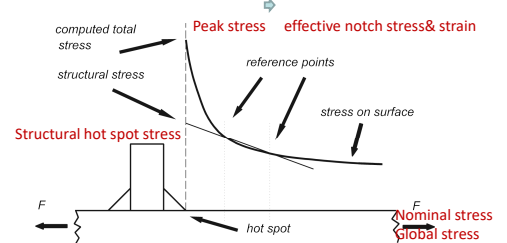
応力とひずみ（弾塑性挙動）

- Sudden changes in the geometry is resulted in stress concentration.



疲労照査に用いられる応力の整理

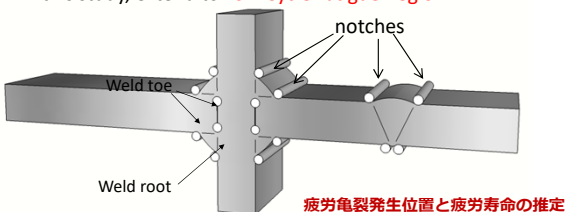
- Nominal stress が定義できない。Hot spot stress
- 溶接ビードの形状はまちまち。
- 溶接Toeの影響は？
- Root Crackは？



Effective notch conceptを適用する

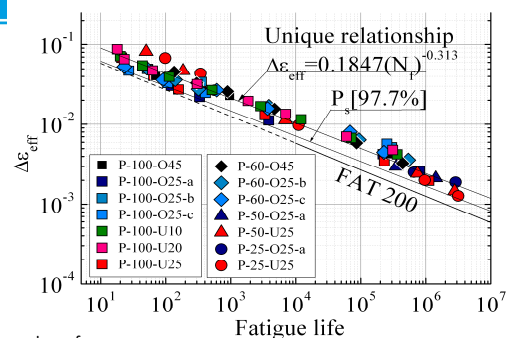
About fictitious notches

- Assign notches of radius 1mm at "weld root" and "weld toe"
- Assessment of fatigue strength in High Cycle Fatigue Region
- In this study, extend to Low Cycle Fatigue Region



Sajprasertkit, Miki et al. Int. J. of Fatigue, 2012 and 2014

Local strain (Effective Notch) approach

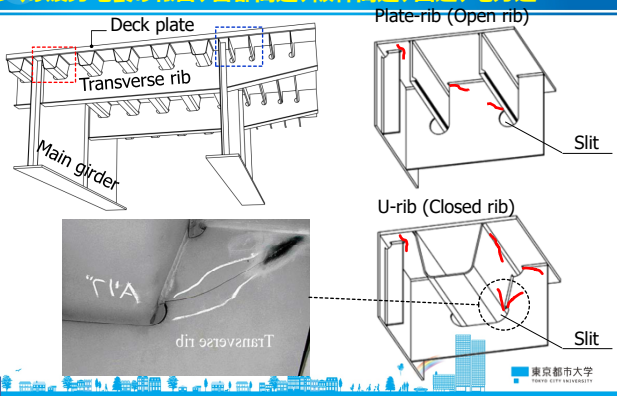


N_f = number of cycles to 20% load drop

Effective notch strain can be used to assess fatigue strength from low to high cycle fatigue region

最近の話題：鋼床版の疲労

1990年一、多くの疲労亀裂の報告、首都高速、阪神高速、国道、地方道

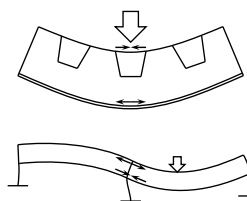


縦リブ横リブ継ぎ手部の変位と変形

- 縦リブ、横リブとも縦内、横外方向に変位する→複雑な変形

設計での照査

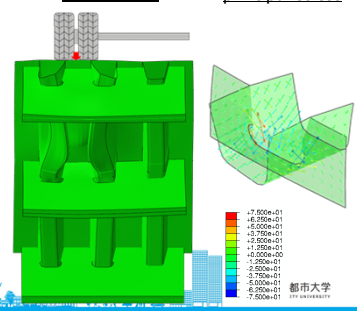
横リブ面内の曲げに対するリブ十字継ぎ手として設計（2002）
標準ディテールで設計（2002～）



FEA result

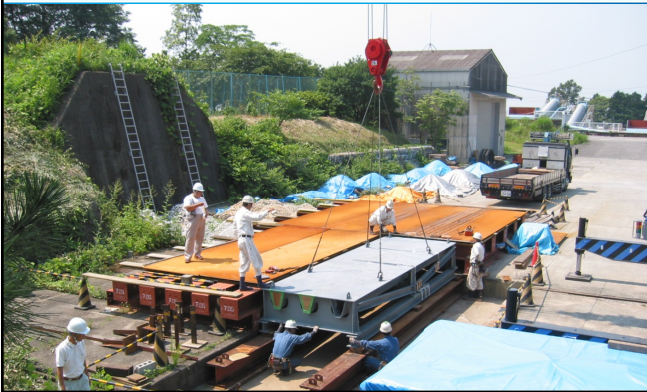
Three dimensional deformation

Connection, principal stress



載荷台での実験：

ディテールを変えた試験パネルを取り付け、トラックを用いて載荷



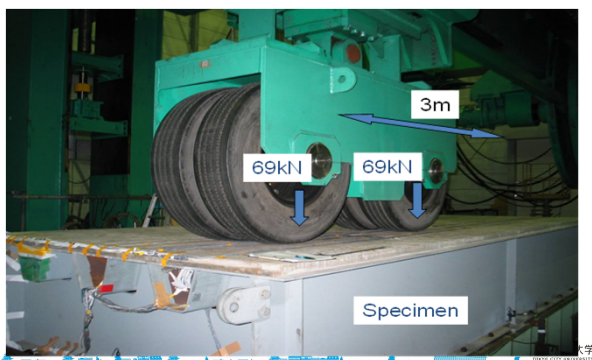
3連ジャッキ：フェーズを変えての疲労試験

移動荷重の効果を見たい



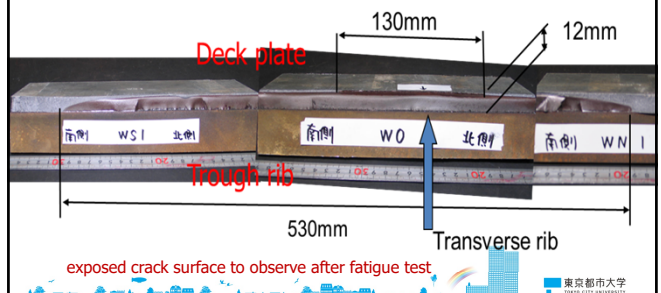
移動荷重試験機による疲労試験

載荷ライン：狙った疲労亀裂を発生させること



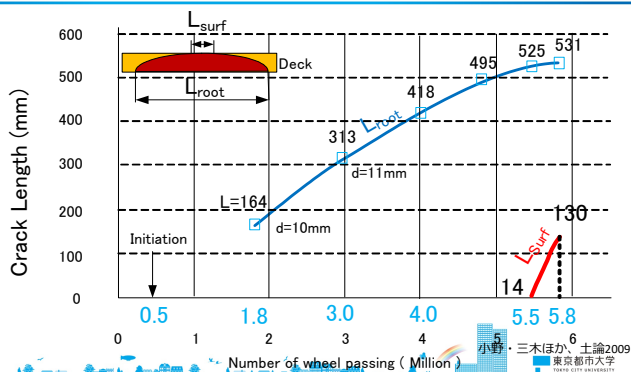
疲労亀裂の進展挙動の観察

疲労亀裂は合体を繰り返しながら、長く長く引き伸ばされるように進展



デッキプレート板厚方向への疲労亀裂の進展

疲労亀裂は50万回で発生し、引き延ばされるように長い亀裂になって進展し、550万回で表面に現れる。さむて特徴的な進展挙動。



Retrofit Works

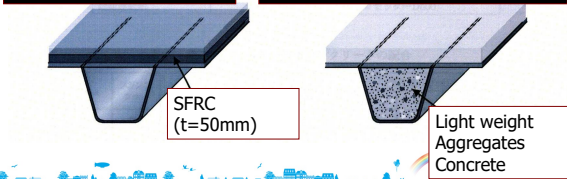
Original detail

Plate Attaching Method



SFRC Covering Method

Concrete Filling in Trough Ribs



疲労対策 合成鋼床版化 SFRCの適用

横浜ハイブリッド下層部（国道357号）、予防保全としての措置
合成鋼床版：名古屋高速の実績、SFRC：湘南大橋でTry

三木、鈴木ほか、土論2006
小野、三木ほか、土論2009

SFRC Pavement
Adhesive
Deck Plate
Stud

デッキプレート周辺
の疲労には有効。
下スリット近傍には？

75mm
12mm

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東京ゲートブリッジ（2012年2月12日開通）設計段階から関与

Total length: 760m,
160m+440m+160m
3 span continuous girder bridge,
Truss – Box Hybrid structure
Apply newly developed
High Performance Steel for Bridge
All welded structure
New OSD

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疲労のポイント：縦リブ、横リブの交差構造

Japan	Standard (Roadway)	Tokyo gate	Hanshin expressway	Railway bridges
Severn				
Millau	George Washington	Golden gate	New Carquinez	Oakland Bay
		Diaphragm		

■ Non-slit connection is possible by devising fabrication

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3-D的な変位と変形をどのように抑え込むのか

交差部のスリットの有無と形状、板厚など
ところでHot Spotはどこ？

Principal
Abs. max.
[N/mm²]

60.0
45.0
30.0
15.0
0.0
-15.0
-30.0
-45.0
-60.0
-75.0
-90.0
-105.0
-120.0

Min
MAX

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Hot-spot location は荷重の移動に伴って移動

Longitudinal
Out-of-plane
Mix of multiple deformations

Stress concentrated

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東京ゲートブリッジ用の高疲労強度鋼床版

デッキプレート板厚16mm、大断面リブ(w=400、内部リブ

菅沼・三木、土論2007

デッキプレートとトラフリブ間の疲労対策

Thickness of deck plate 16mm
Width of long. Ribs 400mm
And 75% penetration

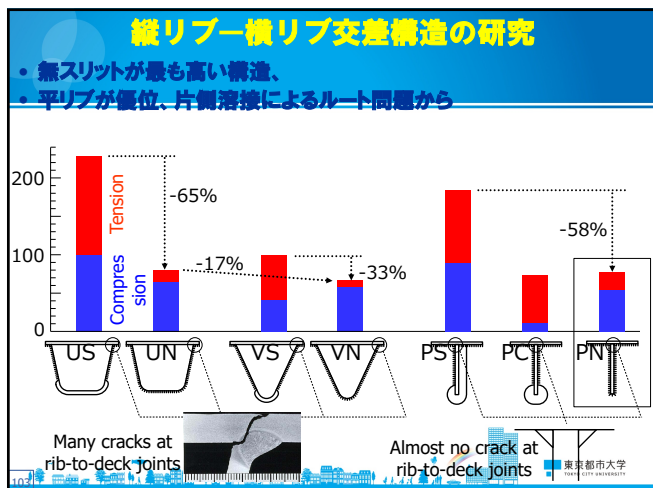
縦リブー横リブ交差部の疲労対策

スリットの形状改善と内部リブ

横リブ間隔4m
鋼重を増さないうで高疲労化

400
16
75%
8
15
R20
343
120

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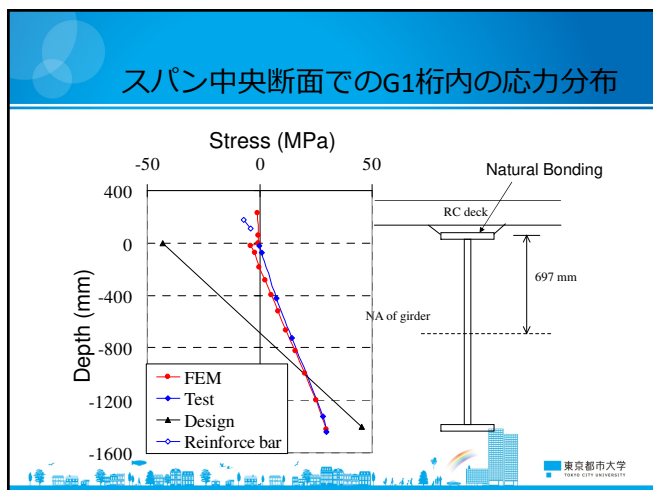
疲労の原因は活荷重と活荷重応力

活荷重の実態
Weigh Stationでの実測

WIM (Weigh in Motion)への関心
BWIMは1980年代 F.Mosesのアイデア、
米国各州で導入
1995年(?) 東名高速で実施
Real time 処理とすることによりMonitoring 研究へ

活荷重応答
実際に生じる応力と設計応力の差：実応力比
疲労設計への導入 (2002年の疲労設計指針)

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モニタリング研究」ー1 光通信網【情報BOX】を利用した 橋梁の健全度長期遠隔モニタリング:2001-

リアルタイム自動測定・自動転送

ひずみ、たわみ、変位、
温度、腐食電流

リアルタイムデータ処理

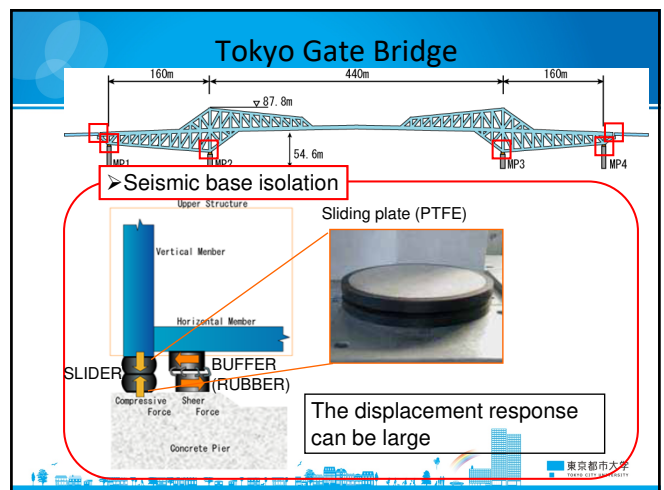
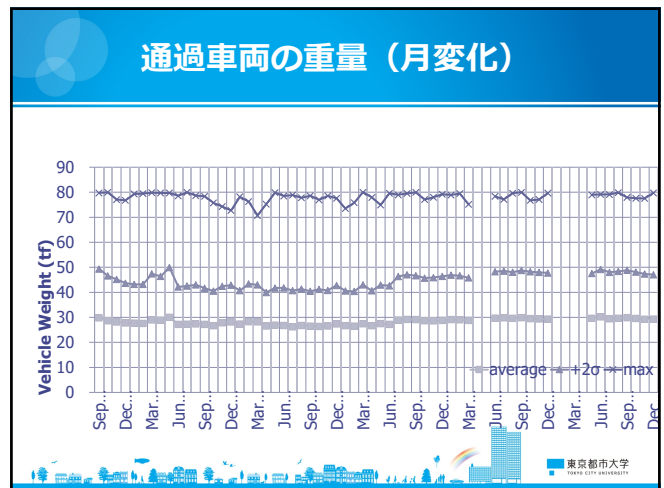
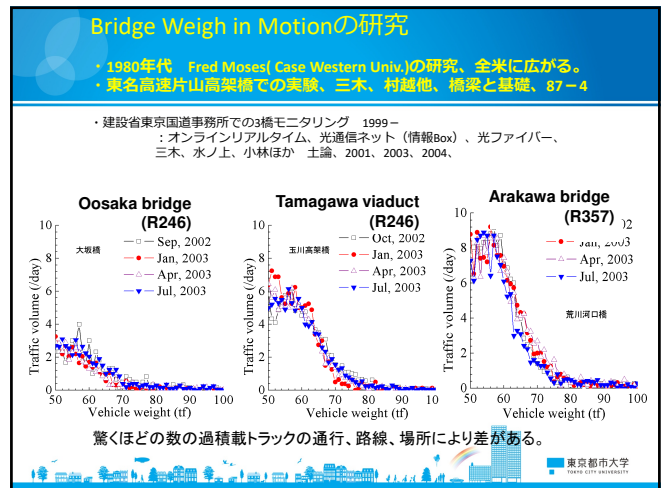
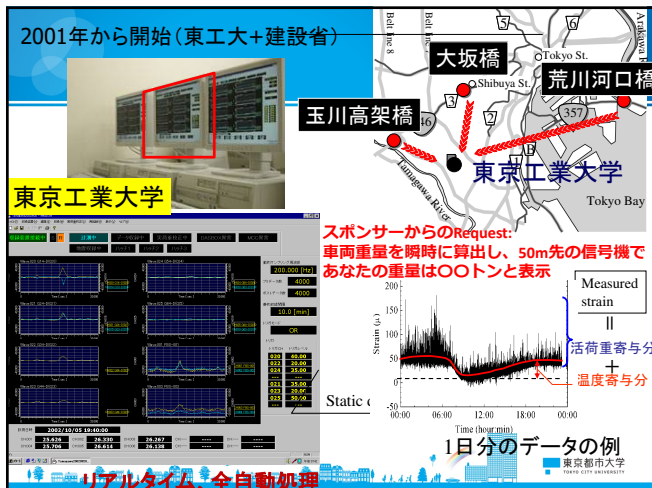
温度寄与分と
荷重寄与分分離

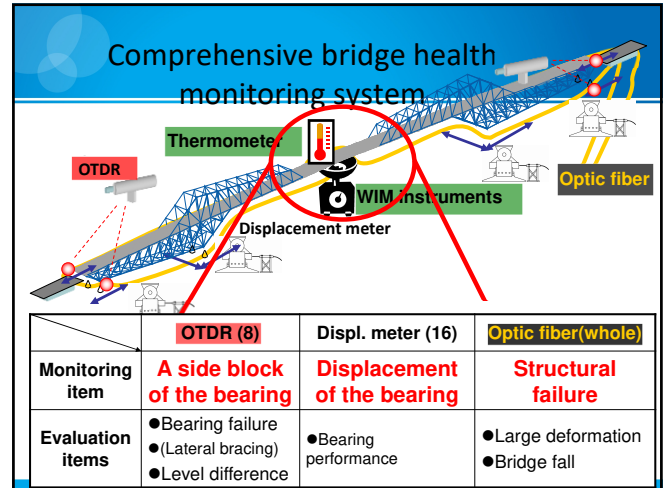
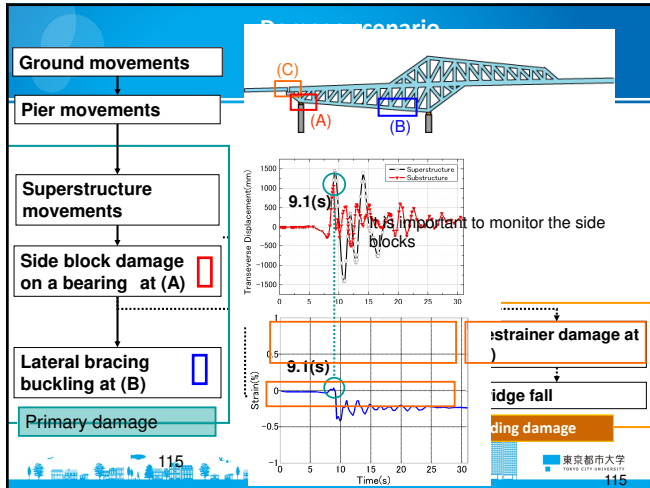
Weigh-in-Motion
温度変形挙動など

荒川河口橋

玉川高架橋

東京都市大学





経済産業省
METI
Journal
6・7月号
June / July 2013

がんばる、センサー

IoTのグループ
MEMSのグループの関心
スマートインフラ
いろいろな団体ができた。

第2特集 “おもてなし経営”とは何か?
Special Report “ダイバーシティ経営企業100選”と“なでしこ経済”

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東京都市大学での研究

中村英夫学長の指示

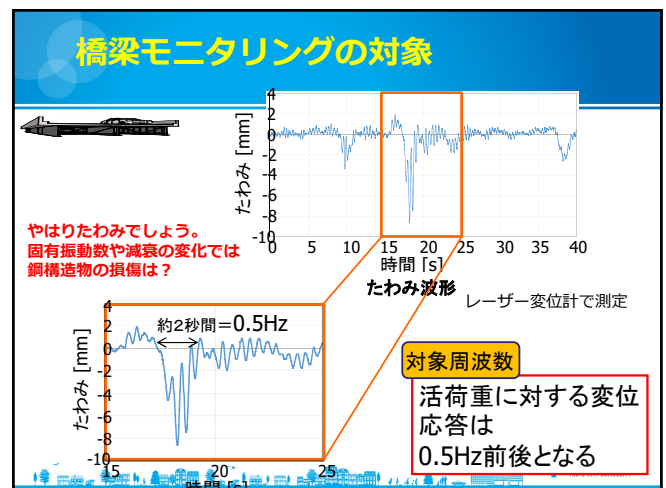
- 総合研究所所属で都市インフラのメンテナンス関係の研究をやるように
- 都市大教員への科研の指導

疲労試験機などの研究設備なし
何をしようか

現場がある。首都高速、新幹線など
スマートインフラ
橋に神経と脳を取り付け、傷を受けたら自分で痛いという
中村英夫学長コメント

Self detection, self Diagnosis

東京都市大学
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科研基盤 (A)の採択：加速度から変位を求めること 振動台実験状況 (sin波、シミュレーション波)

入手可能なセンサー
10タイプの購入し
性能確認する。
0.1-100Hz、
0.1mmの変位を測定

振動台制御用PC 振動台 レーザー変位計

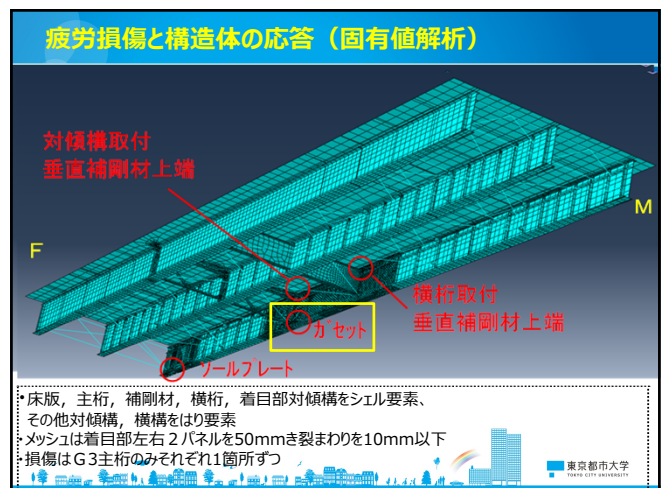
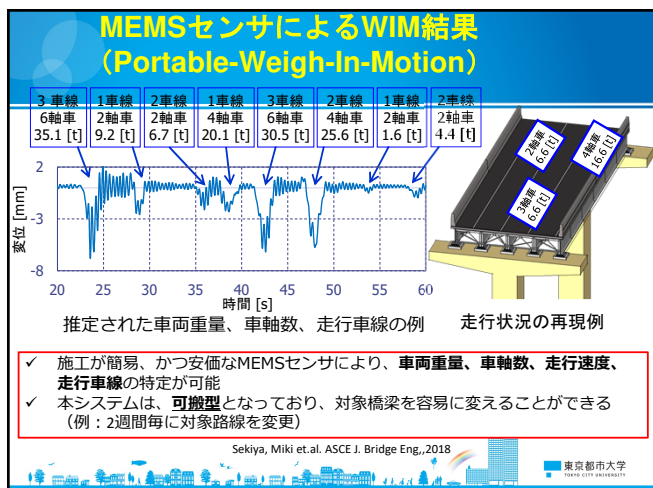
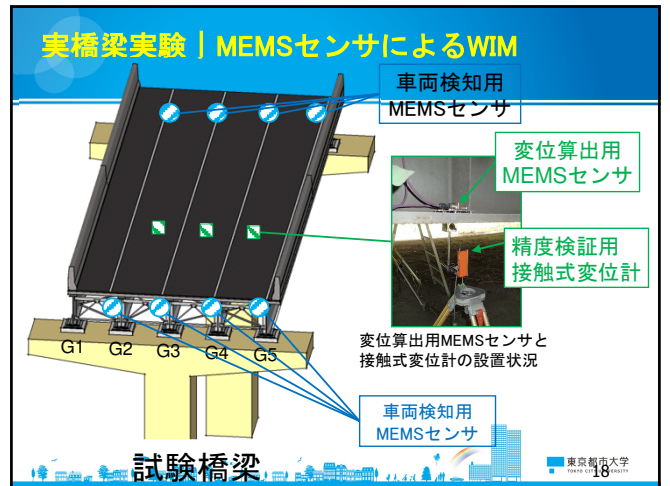
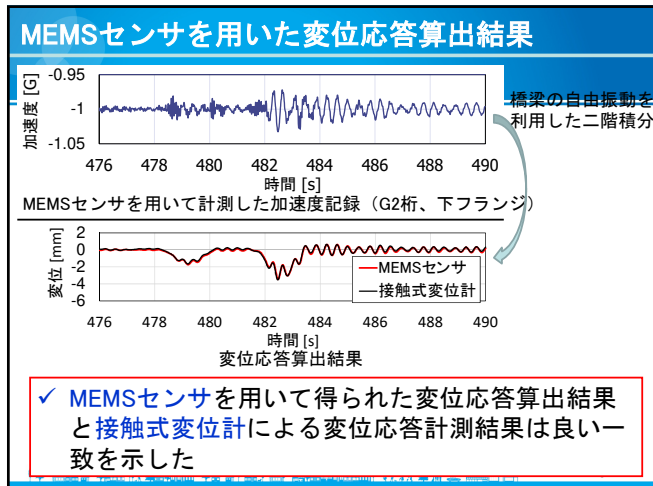
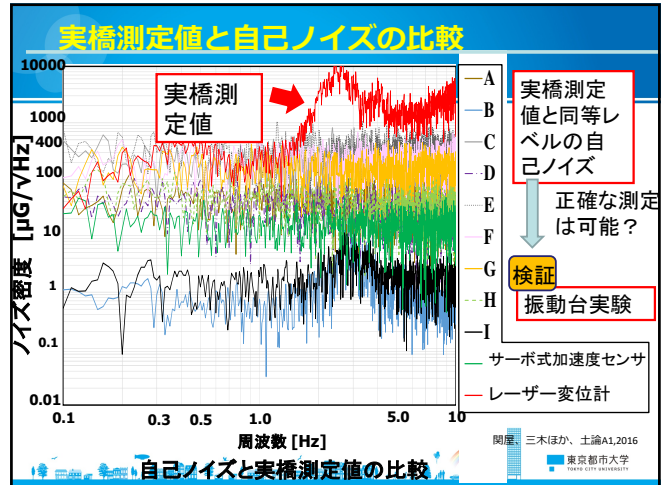
加速度センサ及び
レーザー変位計用PC

加速度センサ

温湿度計

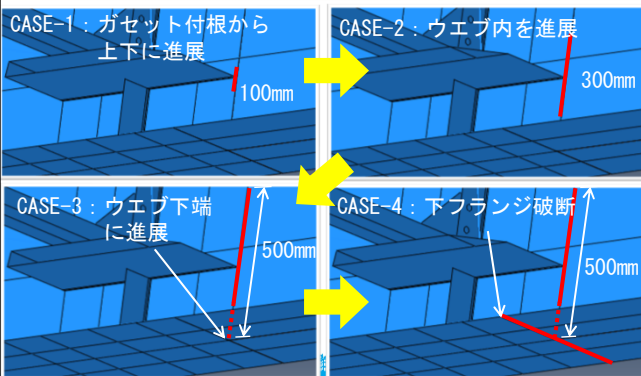
全てのセンサーを
振動台にのせ
変位制御で動かし
加速度を測定

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3. 固有値解析概要

損傷ケース 横構ガセット付け根に生じるき裂



全体モード 振動数 (Hz)	健全	損傷1			損傷2				損傷3		
		CASE1 下フランジ 200mm 進展	CASE2 下フランジ 破断	CASE3 ウェブ 進展	CASE1 ウェブ 200mm 進展	CASE2 ウェブ 400mm 進展	CASE3 ウェブ 500mm ウェブ 下端	CASE4 下フランジ 破断	CASE1 G3 VS 1箇所 破断	CASE2 G3 VS全て 破断	CASE3 橋梁 VS全て 破断
1次:鉛直1次	2.720	2.720	2.719	2.718	2.719	2.719	2.718	2.579	2.720	2.720	2.719
2次:ねじれ1次	4.825	4.820	4.793	4.642	4.825	4.825	4.825	4.768	4.825	4.824	4.819
3次:水平1次	6.962	6.960	6.945	6.878	6.962	6.962	6.962	6.923	6.962	6.955	6.936
4次:鉛直2次	8.534	8.524	8.471	8.229	8.533	8.532	8.532	8.376	8.534	8.533	8.531
5次:ねじれ2次	12.334	12.326	12.275	11.603	12.333	12.330	12.329	11.876	12.334	12.332	12.316
6次:鉛直3次	12.919	12.884	12.729	12.496	12.919	12.918	12.918	12.853	12.917	12.915	12.909
7次:水平2次	13.483	13.476	13.439	13.387	13.482	13.482	13.482	13.492	13.482	13.462	13.424

※着色色部は連成モード等モード形状が変化

Sekiya, Miki et.al J. Sound and Vibration 438, 2019



橋梁の疲労事例についてはData-Baseと2冊の本で公表

206 cases of fatigue failure

多分、世界最大のData-base、世界中からアクセス
日本からはほとんどなし
関心が低い、それとも英語のため使ってください！！

<http://fatigue.civil.tcu.ac.jp/pukiwiki>

おわりに



研究にお付き合いいただいた
市川教授、助手、卒研生、修士学生、博士学生、研究生、研究員に感謝です

ご清聴、ありがとうございました

土木学会講堂

2021年12月16日

第9回 鋼構造技術継承講演会

積雪寒冷地における鋼道路橋の 維持管理と耐震性能向上 への貢献

北海道大学 名誉教授
工学博士 林川俊郎

略歴（自己紹介）

学 歴

昭和47年3月 北海道大学工学部土木工学科 卒業
昭和49年3月 北海道大学大学院修士課程 修了
昭和59年3月 北海道大学工学博士(論文博士) 取得

職 歴

昭和49年4月 北海道大学工学部 助手
昭和60年3月 プリンストン大学(米国) 客員研究員
昭和63年8月 北海道大学工学部 助教授
平成16年4月 北海道大学大学院工学研究科 教授
平成25年4月 北海道大学 名誉教授
平成25年4月 北海道大学大学院工学研究院 特任教授
平成27年3月 北海道大学大学院工学研究院 退職

研究歴（自己紹介）

昭和49年4月～ 北海道大学工学部において
「鋼床版の耐力力とUリブの規格化」
昭和51年4月～ 北海道大学工学部において
「走行荷重による連続桁、吊橋の動的応答」
昭和61年4月～ 北海道大学工学部において
「橋梁構造物の固有振動特性と動的応答」
平成5年4月～ 北海道大学工学部において
「歩道橋の動的応答解析と振動使用性」
平成9年4月～ 北海道大学大学院工学研究科において
「橋梁構造物の大地震時非線形挙動」
平成14年4月～ 北海道大学大学院工学研究科において
「鋼製斜張橋タワーの大地震時非線形挙動」
平成19年4月～ 北海道大学大学院工学研究科において
「免震支承と落橋防止システムの耐震性能」

社会活動（自己紹介）

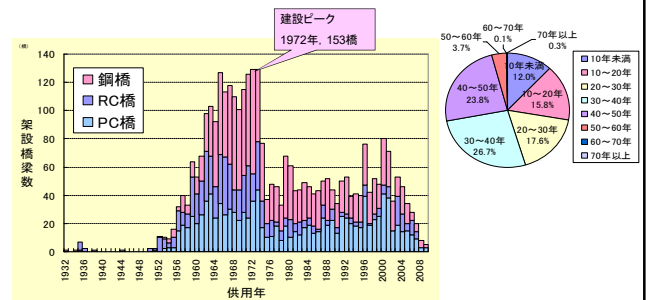
平成5年5月 道路管理技術検討委員会委員
平成5年8月 岩見沢大橋技術検討委員会委員(道)
平成6年10月 白老湾海岸整備計画調査検討委員会(局)
平成7年2月 美原大橋及びその周辺高度利用検討委員会委員長
平成8年11月 札幌市地震対策土木技術検討委員会委員
平成9年9月 道々静内中札内線シタリ11号橋検討委員会委員
平成9年9月 千歳ジャンクションランプ橋事故対策検討委員会委員
平成13年9月 北郷立体交差技術検討委員会委員(局)
平成13年9月 北海道道路管理技術センター道路防災ドクター
平成14年4月 公物の維持管理更新のあり方検討に係る専門員(道)
平成14年9月 (財)土木研究センター落橋防止構造に関する研究委員会
平成15年7月 追直漁港人工島橋梁景観検討委員会委員長(局)
平成15年9月 平成15年台風10号災害調査委員会委員(道)
平成15年9月 平成15年十勝沖地震検討委員会検討委員会委員

社会活動（自己紹介）

平成16年2月 公共土木施設長寿命化検討委員会委員(道)
平成16年4月 札幌市地震防災検討委員会委員
平成16年6月 北海道土木技術会鋼道路橋研究委員会委員長
平成16年10月 追直漁港施設利用検討委員会委員(局)
平成17年3月 創成橋保存技術検討委員会委員長(札幌市)
平成17年12月 ECCを用いた橋面構造に関する検討委員会委員(局)
平成18年6月 北海道新幹線冬季対策検討委員会委員(鉄道運輸)
平成19年4月 札幌市地震被害想定委員会委員
平成20年3月 溶接学会北海道支部支部長
平成20年8月 札幌市橋梁長寿命化修繕計画策定検討委員会委員
平成21年4月 土木学会北海道支部支部長
平成21年4月 北海道開発局道路防災有識者(局)
平成21年5月 札幌自動車道新川高架橋伸縮装置損傷検討委員会
平成21年8月 橋梁長寿命化修繕計画に関する有識者会議委員(局)
平成24年7月 東日本高速道路北海道支社橋梁長寿命化検討委員会

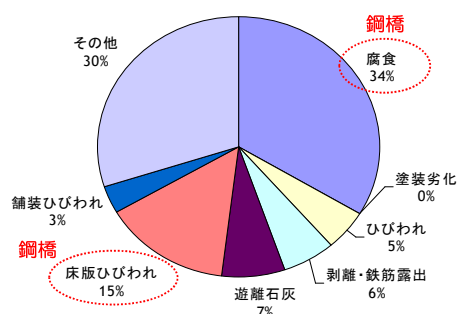
■道内の国道橋の建設推移

約4,000橋 (H18年時点、側道橋も含む)



■北海道の損傷の特徴

北海道の国道橋梁の損傷の特徴は？

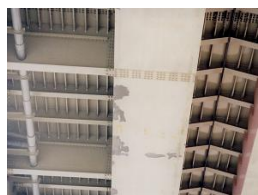


■北海道における鋼橋の損傷について

■鋼橋の損傷

■ 腐 食

第1段階 塗装のはがれ



内部の錆の発生により
塗装がはがれる場合もある

第2段階 錆の発生



■ 腐 食

第3段階 鋼材断面の欠損



腐食が進行すると、鋼部材が腐食により欠損する場合もある。

■ 腐 食

北海道での事例



【支承部近傍】

■ 腐 食

北海道での事例



【桁端部】

■ 腐食

桁端部に腐食・損傷が多い理由として・・・

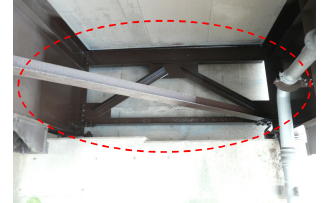
- ① 伸縮装置の損傷・劣化
→ 路面からの土砂・雨水の流れ落ち
土砂・雨水の堆積・滞水
- ② 閉鎖的空間・狭隘（風通しの悪さ）
→ 堆積物の滞留
湿気の滞留
メンテナンス性の悪さ

■ 腐食の補修

桁端部の塗装塗り替えによる補修例
（重防食塗装）・（金属溶射）



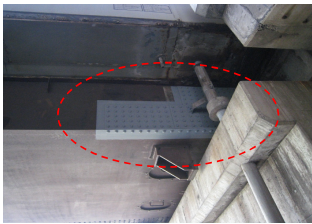
主桁端部塗り替え補修例



端対傾構塗り替え補修例

■ 腐食の補修

当て板による補修例



ボルト接合による当て板補修の例



溶接による当て板補修の例

■ 腐食の補修

橋梁洗浄による塗装の延命化対策例



◇ 予防保全の一つの方法として、橋梁部材に付着した汚れや塩分などを取り除き、できるだけ架設初期の状態に近づけて、橋梁全体の腐食寿命を延ばすことを目的とした対策。

第1章 設計条件WG

■改訂ポイント その他-1

1.3.2 耐候性鋼材の適用地域

p1-15

【改訂ポイント】
適用地域は、道示（Ⅱ鋼橋編）によることを標準とした。

<背景>
従来、道路管理者により適用地域に相違があった。基本方針を統一した。

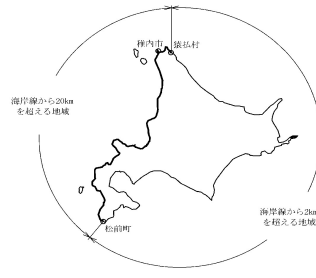


図-第1.3.4 耐候性鋼橋梁の適用地域

■ 亀裂

鋼部材は断面が薄いため、少しの亀裂でも、安全性を損なう可能性がある。



■ 亀 裂

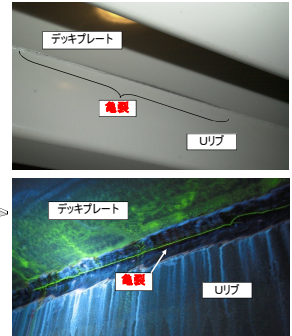
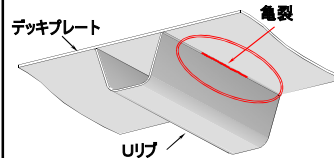


斜材の破断
(コンクリート埋め込み部の
滲水等による腐食の進行)

■ 亀 裂

北海道での事例

鋼床版の損傷

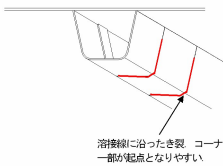


磁粉探傷試験

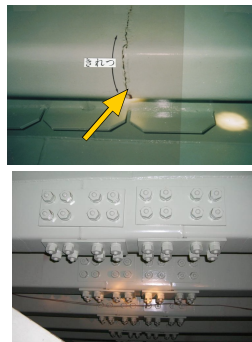
■ 亀 裂

北海道での事例

Uリブ溶接部の損傷



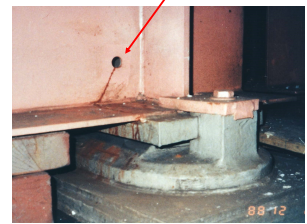
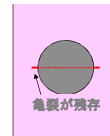
溶接部に沿った亀裂、コーナ
一部が起点となりやすい



高力ボルトを用いた補修例

■ 亀裂の補修

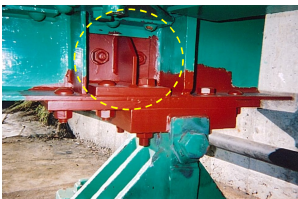
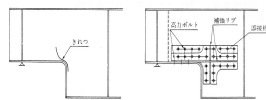
ストップホールによる
補修例 (応急対策)



- ・ 確実に先端に孔をあけることが重要
- ・ グラインダーで削りながら非破壊検査により亀裂先端を発見して開孔

■ 亀裂の補修

補強板添加による補
修例



- ・ 下フランジとウェブを一体化させたI型補強板が望ましい
- ・ できれば新規支承や大きめのソールプレートへの取り替えが望ましい

■ ボルトの脱落

遅れ破壊による高力ボ
ルトの脱落、または、腐食に
よるものなど



□遅れ破壊とは？

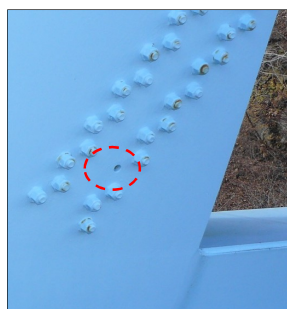
- 遅れ破壊とは、一定の引張荷重が加えられている状態で、ある時間が経過したのち、突然脆性的に破壊する現象です。
- これまでの調査から、F11T高力ボルトの特に昭和46年~52年頃のものに多く見られます。
- また、腐食環境の厳しい箇所に発生しやすい

■ ボルトの脱落

北海道での事例



遅れ破壊が懸念される橋については、高力ボルトの交換が必要となってくる。



■ 床版の損傷

- 鋼橋において、主にコンクリート床版が用いられる。
- 通行車両が繰り返し通過することで、疲労による損傷が問題となる。
- 疲労破壊とは、小さい力であっても、継続的に繰り返し受け続けると、受けた部材の強度が次第に小さくなり、やがて変形し、破壊に至ることをいう。
- 特に、北海道は凍害による損傷（陥没）が多く発生している。

■ 床版の損傷

床版の損傷プロセス

潜伏期
(一方向ひびわれ)



進展期
(二方向ひびわれ)



■ 床版の損傷

床版の損傷プロセス

加速期
(角落ち、漏水)



劣化期
(局部的な陥没)



■ 床版の損傷

北海道の損傷事例

床版破壊事例：2008年6月



床版破壊事例：2008年7月



■ 床版損傷は安全・快適な交通を阻害し、利用者への影響は大きい。そのため、このように事態ならぬよう、日常点検時に十分な配慮が必要である。

■ 床版の損傷

北海道の損傷事例

凍害による損傷

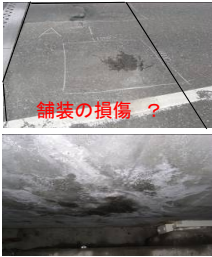


舗装を撤去すると、床版コンクリートのほとんどが砂利化している。また、滞水も確認された。

■ 床版の損傷

北海道の損傷事例

点検時の注意



下面の状況：漏水はあるがひび割れの進展が無い

舗装を撤去してみると、床版上面が砂利化、人力でもザクザクの状態



■ 床版の損傷

北海道の損傷事例

水の影響とは

- 乾燥収縮や交通荷重の繰返しによりひび割れが発生する
- ひび割れから水の浸入しひび割れ面の摩耗が促進する
- 浸透水によって凍結融解が促進する
- 上記現象によって・・・
- ひび割れ面のこすり合わせ現象とひび割れ面内の水が路面に吹き出すポンピング現象が生じる。
- 床版表面のコンクリートはセメントペーストと骨材に分離され骨材化現象が生じる。

■ 支承の損傷について

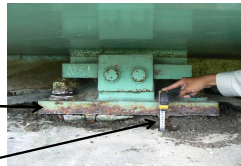
■ 支承

- 支承は、橋台、橋脚上に設置され、上部構造と下部構造をつなぐ役割をもつ。
- 支承(沓)の種類
 - 鋼製
 - ゴム製
- 支承の設置
 - 上部構造とはボルトで直接連結
 - 下部構造ともボルトで連結されるが、下部構造上に、台座コンクリートとモルタル(沓座モルタル)を設け、その上に支承が設置されている。



■ 支承の損傷

- 支承本体
 - 腐食
 - 欠損
- 台座コンクリート
 - 破損
- 沓座部
 - 表面コンクリートの劣化
 - 土砂堆積



第12章 付属物

■改訂ポイント【支承】

写真	説明	改訂ポイント	写真	説明	改訂ポイント
	図-12-2-1 支承部の損傷事例(その1)	・腐食 ・欠損		図-12-2-2 支承部の損傷事例(その2)	・腐食 ・欠損
	・腐食 ・欠損			・腐食 ・欠損	
	・腐食 ・欠損			・腐食 ・欠損	
	・腐食 ・欠損			・腐食 ・欠損	
	・腐食 ・欠損			・腐食 ・欠損	
	・腐食 ・欠損			・腐食 ・欠損	
	・腐食 ・欠損			・腐食 ・欠損	
	・腐食 ・欠損			・腐食 ・欠損	
	・腐食 ・欠損			・腐食 ・欠損	

第12章 付属物

■改訂ポイント 【支承】

12.2.4 支承の補修および交換 (pp12-8~12-11)

参考) ゴム支承表面クラックの補修の一例

①地震により大きなせん断変位が残ったゴム支承の補修例



写一解 12.2.1 震災後の支承

写一解 12.2.2 クラックの拡大

写一解 12.2.3 補修後

■伸縮装置・防護柵の 損傷について

■伸縮装置・防護柵

□ 伸縮装置・防護柵ともに、損傷劣化（腐食等）が多く確認される部位。

・伸縮装置

➢ 橋の端部において、桁の温度による伸び縮みを吸収するために設けられる構造

・防護柵

➢ 車両や歩行者の路外への逸脱を防止するために設けられる柵

■伸縮装置

荷重支持式



鋼フィンガー型

鋼ビーム型

鋼製簡易取付横型



簡易荷重支持式

突合せ式

ノージョイント式

鋼製簡易取付縦型

■伸縮装置の損傷

北海道の損傷事例



■錆発生



■止水部材脱落



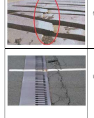
■除雪機接触損傷



寒冷地特有の損傷状況に対応した伸縮装置の改善提案

第12章 付属物

■改訂ポイント 【伸縮装置】

状況	損傷事例	損傷の概要	損傷の対策	状況	損傷事例	損傷の概要	損傷の対策
中央防風壁		へこみ ひび割れ 長さ約3.0m	縦筋のほとんどが折断 縦筋がスリット状に変形 破断し腐蝕が促進 腐蝕が進行	縦筋のほとんどが折断 縦筋がスリット状に変形 破断し腐蝕が促進 腐蝕が進行	中央防風壁		縦筋のほとんどが折断 縦筋がスリット状に変形 破断し腐蝕が促進 腐蝕が進行
	中央防風壁の縦筋	中央防風壁の縦筋 の破断・変形が 確認されている。	中央防風壁の縦筋 の破断・変形が 確認されている。	中央防風壁の縦筋 の破断・変形が 確認されている。			
バム受留部の縦筋		縦筋が 折断	縦筋が折断 縦筋が折断 縦筋が折断	縦筋が折断 縦筋が折断 縦筋が折断	バム受留部の縦筋		縦筋が折断 縦筋が折断 縦筋が折断
	バム受留部の縦筋	バム受留部の縦筋 の破断・変形が 確認されている。	バム受留部の縦筋 の破断・変形が 確認されている。	バム受留部の縦筋 の破断・変形が 確認されている。			
バム受留部の縦筋		縦筋が 折断	縦筋が折断 縦筋が折断 縦筋が折断	縦筋が折断 縦筋が折断 縦筋が折断	バム受留部の縦筋		縦筋が折断 縦筋が折断 縦筋が折断
	バム受留部の縦筋	バム受留部の縦筋 の破断・変形が 確認されている。	バム受留部の縦筋 の破断・変形が 確認されている。	バム受留部の縦筋 の破断・変形が 確認されている。			

第6章 付属物

■改訂ポイント【排水設備】

要求性能(pp6-57~58)

(1) 排水性

(2) 耐荷性

(3) 耐腐食性

(4) 維持管理性

写真6-5-1 舗装面下の排水

写真6-5-2 舗装面下の排水

写真6-5-3 排水設備の設置

写真6-5-4 排水設備の設置

写真6-5-5 排水設備の設置

写真6-5-6 排水設備の設置

写真6-5-7 排水設備の設置

写真6-5-8 排水設備の設置

写真6-5-9 排水設備の設置

写真6-5-10 排水設備の設置

写真6-5-11 排水設備の設置

写真6-5-12 排水設備の設置

写真6-5-13 排水設備の設置

写真6-5-14 排水設備の設置

写真6-5-15 排水設備の設置

写真6-5-16 排水設備の設置

写真6-5-17 排水設備の設置

写真6-5-18 排水設備の設置

写真6-5-19 排水設備の設置

写真6-5-20 排水設備の設置

写真6-5-21 排水設備の設置

写真6-5-22 排水設備の設置

写真6-5-23 排水設備の設置

写真6-5-24 排水設備の設置

写真6-5-25 排水設備の設置

写真6-5-26 排水設備の設置

写真6-5-27 排水設備の設置

写真6-5-28 排水設備の設置

写真6-5-29 排水設備の設置

写真6-5-30 排水設備の設置

写真6-5-31 排水設備の設置

写真6-5-32 排水設備の設置

写真6-5-33 排水設備の設置

写真6-5-34 排水設備の設置

写真6-5-35 排水設備の設置

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写真6-5-43 排水設備の設置

写真6-5-44 排水設備の設置

写真6-5-45 排水設備の設置

写真6-5-46 排水設備の設置

写真6-5-47 排水設備の設置

写真6-5-48 排水設備の設置

写真6-5-49 排水設備の設置

写真6-5-50 排水設備の設置

写真6-5-51 排水設備の設置

写真6-5-52 排水設備の設置

写真6-5-53 排水設備の設置

写真6-5-54 排水設備の設置

写真6-5-55 排水設備の設置

写真6-5-56 排水設備の設置

写真6-5-57 排水設備の設置

写真6-5-58 排水設備の設置

写真6-5-59 排水設備の設置

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写真6-5-62 排水設備の設置

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写真6-5-68 排水設備の設置

写真6-5-69 排水設備の設置

写真6-5-70 排水設備の設置

写真6-5-71 排水設備の設置

写真6-5-72 排水設備の設置

写真6-5-73 排水設備の設置

写真6-5-74 排水設備の設置

写真6-5-75 排水設備の設置

写真6-5-76 排水設備の設置

写真6-5-77 排水設備の設置

写真6-5-78 排水設備の設置

写真6-5-79 排水設備の設置

写真6-5-80 排水設備の設置

写真6-5-81 排水設備の設置

写真6-5-82 排水設備の設置

写真6-5-83 排水設備の設置

写真6-5-84 排水設備の設置

写真6-5-85 排水設備の設置

写真6-5-86 排水設備の設置

写真6-5-87 排水設備の設置

写真6-5-88 排水設備の設置

写真6-5-89 排水設備の設置

写真6-5-90 排水設備の設置

写真6-5-91 排水設備の設置

写真6-5-92 排水設備の設置

写真6-5-93 排水設備の設置

写真6-5-94 排水設備の設置

写真6-5-95 排水設備の設置

写真6-5-96 排水設備の設置

写真6-5-97 排水設備の設置

写真6-5-98 排水設備の設置

写真6-5-99 排水設備の設置

写真6-5-100 排水設備の設置

■ 防護柵の損傷

・ 腐食、亀裂、欠損、破断、凍害





現行基準に則した新規防護柵への取替補修提案

北海道における鋼道路橋の設計及び施工指針の改訂

北海道土木技術会鋼道路橋研究委員会
(平成24年1月 指針の大幅な改訂作業を実施)

初版：昭和44年3月

第2版：昭和47年4月

第3版：昭和54年4月 (P189)

第4版：昭和58年8月 (P215)

第5版：平成元年11月 (P284)

第6版：平成7年12月 (P318)

第7版：平成24年1月 (本改訂)

第1編：設計・施工編 (P390)

第2編：維持管理編 (P386)

第3編：資料編 (P91)

1995年1月17日5時46分 内陸直下型地震

兵庫県南部地震の被害報告書

平成7年

橋梁構造物の地震被害調査派遣

兵庫県南部地震被害報告書













北海道土木技術会 鋼道路橋研究委員会
振動小委員会

ビルツ橋、神戸大橋、ランプ橋

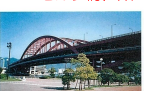
座屈、塑性ヒンジ

























西宮港大橋

鉄道高架橋・ラーメン隅角部

神戸市庁舎・外壁

上部構造・橋脚・建物の壊滅的な崩壊

















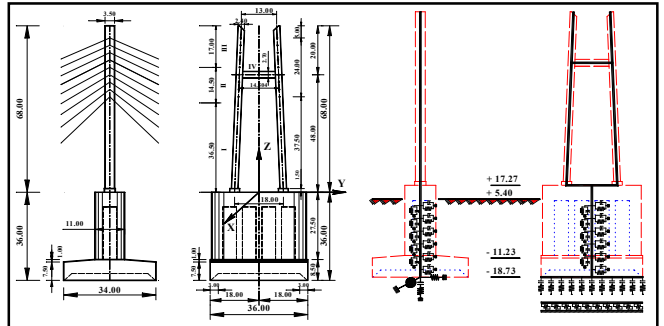
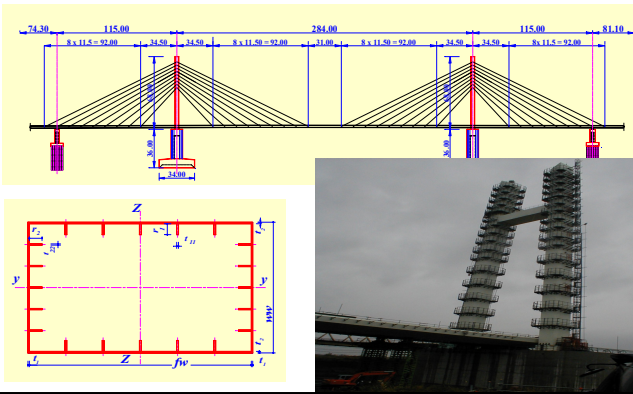








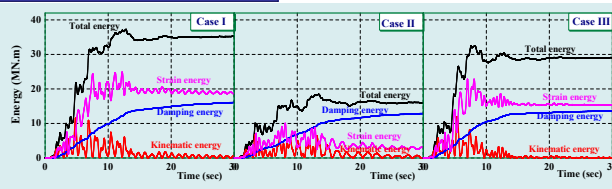
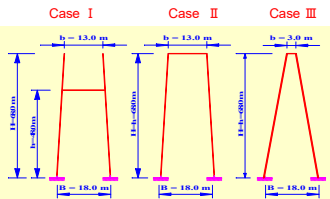
Description of cable stayed bridge and Tower structure model



- ・ レベル2地震動（JR鷹取駅記録地震波、動的相互作用）
- ・ 最悪の損傷シナリオ（安全性、使用性、残留変位etc）
- ・ エネルギー吸収機構（損傷部材、低降伏点鋼、免震・制震）
- ・ 耐震性能向上対策（リダンダンシー：冗長性、記憶合金使用）

Effects of Tower Modal Shapes

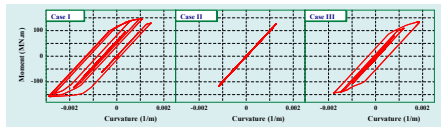
Mode type	Tower modal shapes		
	Case I	Case II	Case III
H ₁	2.072	2.928	1.727
L ₁	0.934	0.935	0.935
T ₁	0.773	0.761	0.543



Effects of Tower Modal Shapes

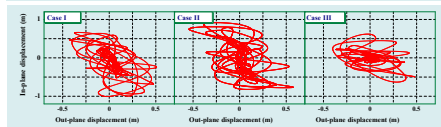
(a)

Moment & curvature relations at tower base



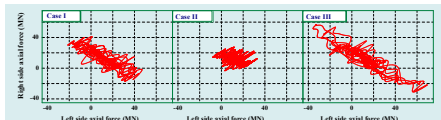
(b)

Trajectory of response
sway displacement of
tower top



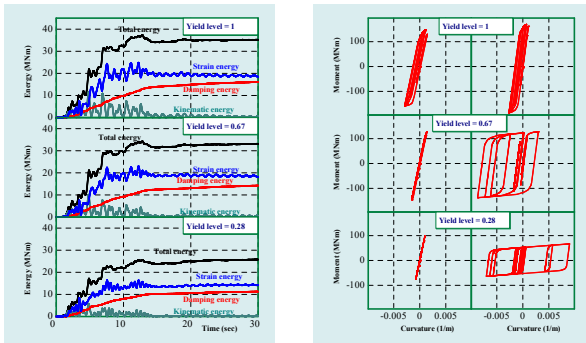
(c)

Axial force relation at tower bases



Dynamic Response of Tower with Low yield material

Low yield material energy dissipation system



Energies time history of tower structure

(a) At tower base (b) At horizontal beam end

落橋防止システムと機能分散型支承

ライフライン施設として道路橋の大地震時における使用限界状態について

（１）落橋防止システムの正常な状態

（２）落橋防止システムの異常な状態

上部構造の異常な配列

曲線床版の幾何学的な形状

- ✓ 複雑な床版の回転
- ✓ 橋軸直角方向への変形

上部構造の異常な連続性

- ✓ けたかかり長
- ✓ 桁間衝突
- ✓ 伸縮装置の遊間

上部構造と下部構造間の

異常な連結状態

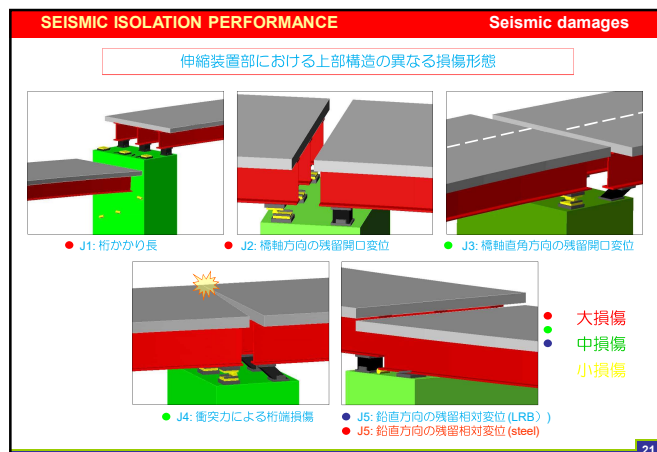
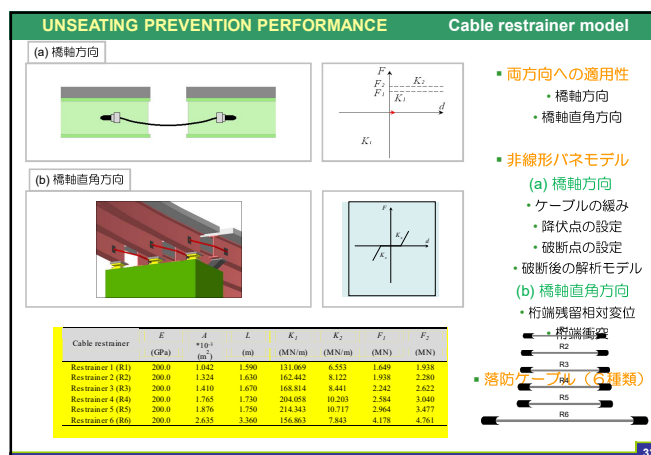
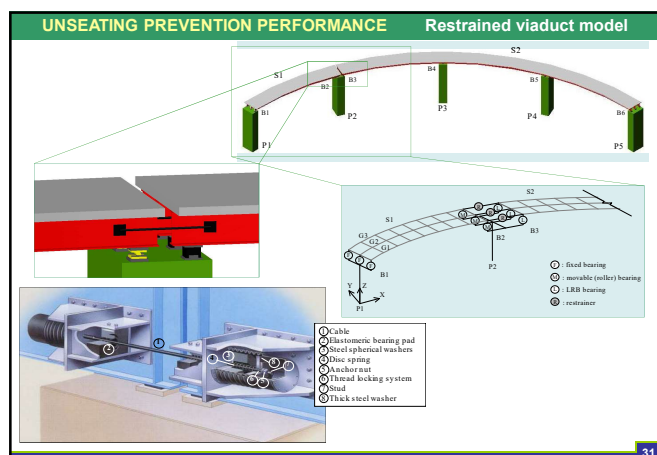
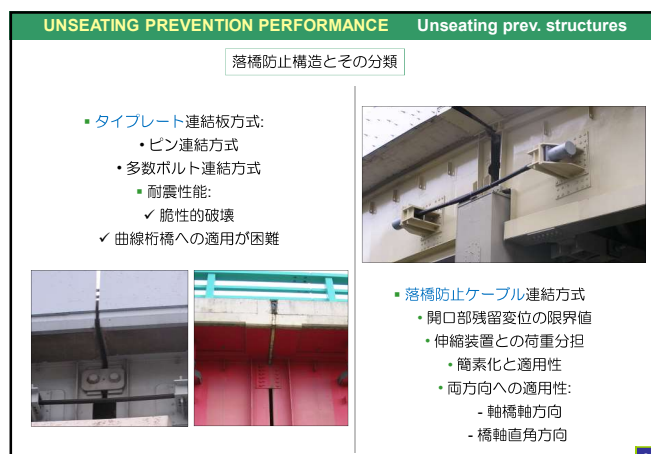
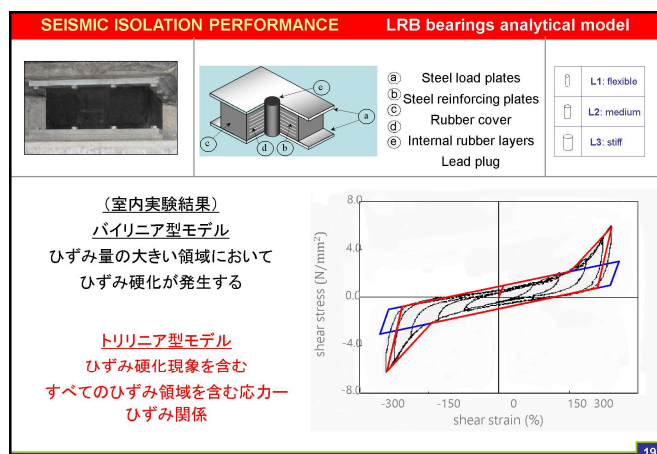
- ✓ 支承部の損傷
- ✓ 鉛直方向の段差



落橋防止防止システム
の異常な状態

通行止め

- ・救急車両等の通行障害
- ・架け替えや補修にかかる直接的なコスト負担
- ・通行止めによる社会に与える間接的なコスト負担



PARAMETRIC STUDY ON STEEL TOWER SEISMIC RESPONSE OF CABLE-STAYED BRIDGES UNDER GREAT EARTHQUAKE GROUND MOTION

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Analytical parametric study on dynamic characteristics of steel tower of cable-stayed bridges is performed to investigate the individual influence of different design aspects, such as damping mechanism, input ground motion, allowable initial construction imperfections, energy dissipation and tower modal shapes. The results show that the horizontal beam height and length and the low yield energy dissipation system significantly affect tower structural behavior. The initial imperfections within design range have slight detrimental effects on the tower seismic response but these effects grow rapidly beyond the design range. Mass proportional damping leads to overestimate tower axial forces and acceleration response.

Key Words: steel tower, cable-stayed bridge, energy dissipation, seismic design, imperfections

1. INTRODUCTION

In recent decades, long span bridges such as cable-stayed bridges have gained much popularity due to their aesthetic appearance, efficient utilization of structural materials, increase of the horizontal navigation clearances and the economic trade off of span length cost of deep water foundation. The trend nowadays for cable-stayed bridges is to use more shallow or slender stiffening girders combined with increasing span lengths. This structural synthesis provides a valuable environment for the nonlinear behavior due to material nonlinearities and geometrical nonlinearities of the relatively large deflection of the structure on the stresses and forces^{1) - 3)}. The Hyogoken-Nanbu earthquake of 17th January 1995, led to an increased awareness concerning the response of highway bridges subjected to earthquake ground motions, the ductility design and dynamic analyses have been reconsidered by Japan Road Association⁴⁾. The necessity has arisen to develop more efficient analysis procedures that can lead to a thorough understanding and a realistic prediction of the precise three-dimensional nonlinear dynamic response of bridge structural systems to improve the bridges seismic performance, to provide damage control and post-earthquake functionality.

In the analysis and design of earthquake resistant structures, particularly bridge structures, the vertical ground motion tends, in general, to be

ignored or underestimated in seismic analysis. The current seismic codes recommend a vertical spectrum with values that vary from half to three quarters of that of the horizontal spectra. This approach seems to be un-conservative in light of ground motion measurements during recent earthquakes, which indicate that the vertical acceleration could reach values even higher than that of the horizontal acceleration. Moreover, in a near field region, the peak of vertical to horizontal spectral ratio is even larger than that of the peak ground acceleration, especially at short periods⁵⁾. Also field observations proved that many structures experienced significant damage attributable to high vertical forces^{6) - 8)}. The dynamic analysis of bridge column and pier structures subjected to horizontal and vertical excitations was considered only recently by some researchers^{8), 9)}, where the results of that analyses with the inelastic plane stress elements, displayed unstable hysteresis loops and little energy dissipation.

The strength and ductility of a thin-walled steel member are particularly sensitive to initial imperfections including geometric imperfection and longitudinal residual stresses. The initial imperfections of these structures that have not been subjected to damage usually result from the fabrication process. Some studies^{10) - 14)} have been carried out on welded and hot rolled structures, it was reported that the detrimental effects of geometric imperfection and welding residual

stresses that led to an appreciable reduction in load carrying capacity. However, a study of the effect of initial imperfections on the dynamic behavior of thin-walled structures is not available in literature. Moreover, the dynamic response of steel tower depends to large extent on tower modal shapes¹⁵⁾, including the horizontal beam position and its elements relative strength. For economical earthquake resistant of steel tower and its protection from the earthquake hazard, the tower structure should be constructed to dissipate a large amount of input seismic energy that could be achieved by proper selection of tower shapes and its elements relative strength. The low strength steels with low yield stresses and large ductility have been introduced for the hysteretic damper concept; the energy dissipation of hysteretic dampers through their materials could be used for structure damage control under large earthquake excitations^{16)~18)}.

A parametric study on steel tower of cable-stayed bridges is performed for investigation of the individual influence of different design aspects on tower dynamic characteristics. This study aims at clarifying the characteristics of the vertical ground motion and damping mechanism effects on critical seismic response quantities of the steel tower under strong ground motions. A comparison of the response with and without the vertical component is performed for two different cases of spectral damping schemes. It is confirmed from numerical results that the vertical excitation could have a detrimental effect on axial force and overestimated acceleration of tower seismic response for the mass proportional damping scheme, but slightly effects for Rayleigh's damping.

Moreover, this study investigates the steel tower seismic response under initial construction imperfections using finite element method. The amplitude of imperfections in the lowest eigenmodes of vibration is considered to be sufficient to characterize the influential imperfections¹⁹⁾. As a result, a great deal of insight has been obtained on the dynamic response of steel tower. The results indicate that both the initial geometric imperfection and the welding induced residual stresses within their design range slightly affect on the tower seismic response but beyond the design range, these effects are characterized by rapidly and nonlinearly growth as the initial imperfections approach severe values. The residual stress effects are due to decreasing plastic deformation capability, consequently promoting brittle behavior.

An effective energy dissipation concept is suggested by a typically concentration of inelastic behavior at tower horizontal beam using low

relative strength and stiffness, which can be achieved by inserting low yield material instead of cross section dimensions reduction. Since the horizontal beam is easy to inspect and repair if necessary, the rest of the structure will remain elastic, thus eliminating permanent damage and minimizing the extent of retrofit. The calculated results prove the effectiveness of the proposed energy dissipation system in reducing structural elements forces and control tower maximum displacement, and enable to determine the optimum position of horizontal beam for economical earthquake resistant design.

2. NONLINEAR DYNAMIC ANALYSIS PROCEDURES

The governing nonlinear dynamic equation of the tower response can be derived by the principle of energy that the external work is absorbed by the work of internal, inertial and damping for any small admissible motion that satisfies compatibility and boundary conditions. By assembling the element dynamic equilibrium equation for the time $t + \Delta t$ over all the elements, the incremental FEM dynamic equilibrium equation^{1), 20)} can be obtained as:

$$[M]\{\ddot{u}\}^{t+\Delta t} + [C]\{\dot{u}\}^{t+\Delta t} + [K]^{t+\Delta t}\{\Delta u\}^{t+\Delta t} = \{F\}^{t+\Delta t} - \{F\} \quad (1)$$

where $[M]$, $[C]$ and $[K]^{t+\Delta t}$ are the system mass, damping and tangent stiffness matrices at time $t + \Delta t$, the tangent stiffness considers the material nonlinearities through bilinear elastic-plastic constitutive model incorporating a uniaxial yield criteria and kinematic strain hardening rule. $\ddot{u}$, $\dot{u}$ and Δu are the accelerations, velocities, and incremental displacements at time $t + \Delta t$, respectively, $\{F\}^{t+\Delta t} - \{F\}^t$ is the unbalanced force vector. The dynamic equilibrium equation of motion considers both geometrical and material nonlinearities that affect the tangent stiffness and internal forces calculation.

In this study, the Newmark step-by-step integration method is used for the integration of equation of motion, since it has been experienced that the Newmark's β method is the most suitable for nonlinear analysis; it has the lowest period elongation and has no amplitude decay or amplifications. In addition, the stability concern is not a problem with the variable ratio of time increment to natural period. The algorithm is unconditionally stable if $\beta \geq (\gamma + 0.5)^2/4$. In this study, the Newmark's β of constant acceleration scheme for the solution of the differential equation

of motion is considered for which β is equal to 0.25. The second numerical parameter γ of Newmark's β method is set as $\gamma = 0.5$ to avoid a superfluous damping in the system. The equation of motion is solved for the incremental displacement using the Newton-Raphson iteration method; the stiffness matrix is updated at each increment to consider the geometrical and material nonlinearities and to speed the convergence rate.

3. FINITE ELEMENT OUTLINE

(1) Finite element model

The steel tower of a three span continuous cable-stayed bridge located in Hokkaido, Japan is considered, in which the main span length is equal to 284m. The steel tower is taken out of the cable-stayed bridge and modeled as three-dimensional frame structure. A fiber flexural element is developed for characterization of the steel tower and that element incorporates both geometric and material nonlinearities. The Hermitian cubic displacement field is employed for the transverse bending displacements of the element and a linear displacement field is employed for the axial and torsional displacements. The stress-strain relationship of the beam element is modeled as bilinear stress strain relation for the beam column element. The yield stress and the modulus of elasticity are equal to 355 MPa (SM490Y) and 200GPa, respectively, the strain hardening in the plastic region is equal to 0.01.

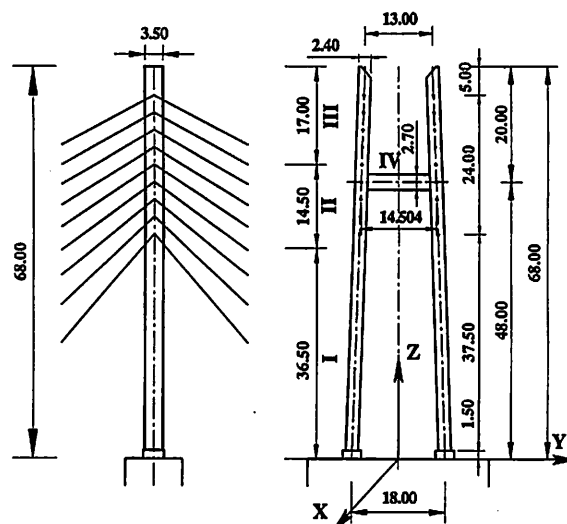
Inelasticity of the fiber flexure element is accounted for by the division of the cross section into a number of fiber zones with uniaxial plasticity defining the normal stress-strain relationship for each zone, the element stress resultants are determined by integration of the fiber zone stresses over the cross section of the element. By tracking the center of the yield region, the evolution of the yield surface is monitored, and a stress update algorithm is implemented to allow accurate integration of the stress-strain constitutive law for strain increments, including full load reversals. To ensure path dependence of the solution, the implementation of the plasticity model for the implicit Newton-Raphson equilibrium iterations employs stress integration, whereby the element stresses are updated from the last fully converged equilibrium state. The transformation between element local and global coordinate systems is accomplished through a vector translation of element forces and displacements based on the direction cosines of the current updated element coordinate system.

The nonlinear behavior of cable elements is idealized by using the equivalent modulus approach, in this approach each cable is replaced by a truss element with equivalent tangential modulus of elasticity E_{eq} that is given by Ernst²¹ as:

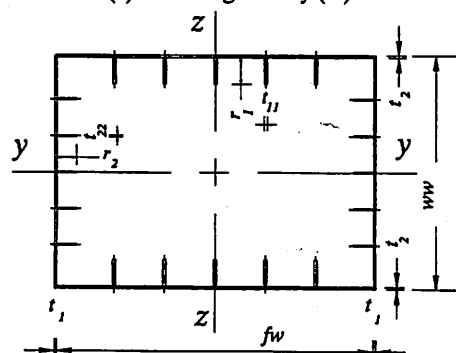
$$E_{eq} = E / \{1 + EA (wL)^2 / 12T^3\} \quad (2)$$

where E is the material modulus of elasticity, L is the horizontal projected length of the cable, w is the cable weight per unit length, A is the cable cross sectional area and T is the cable pretension force. It can be noticed that the nonlinearity of the cable stays originates with an increase in the loading followed by a decrease in the cable sag as a consequence the apparent axial stiffness of the cable increases. The inclined cable is represented by an equivalent straight cable element with relative axial deformation (Δl), the stiffness matrix of the cable element K has the value equal to $E_{eq}A/l$ for $\Delta l > 0$, and the cable stiffness vanishes and no element force exist when shortening occurs, i.e. $\Delta l < 0$.

This cable-stayed bridge has nine cables in each tower side. The dead load of the stiffening girder is considered to be equivalent to the vertical component of the pretension force of the cables and acted vertically at their joints. The inertia forces



(a) Tower geometry (m)



(b) Cross section

Fig. 1 Steel tower of cable-stayed bridge

Table 1 Cross section dimensions of different tower parts (cm)

Tower parts	Outer dimension				Stiffener dimension			
	f_w	w_w	t_1	t_2	r_1	r_2	t_{11}	t_{22}
I	240	350	2.2	3.2	25	22	3.6	3.0
II	240	350	2.2	3.2	22	20	3.2	2.8
III	240	350	2.2	2.8	20	20	2.8	2.2
IV	270	350	2.2	2.6	31	22	3.5	2.4

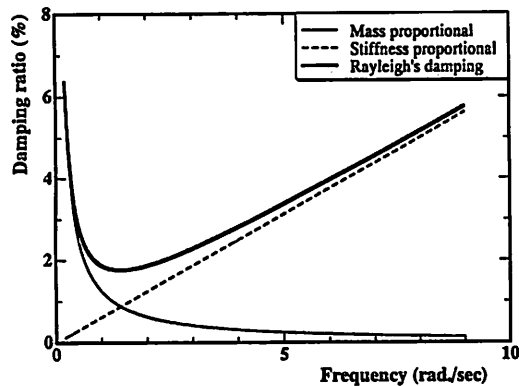


Fig. 2 Damping ratio and frequency relationship

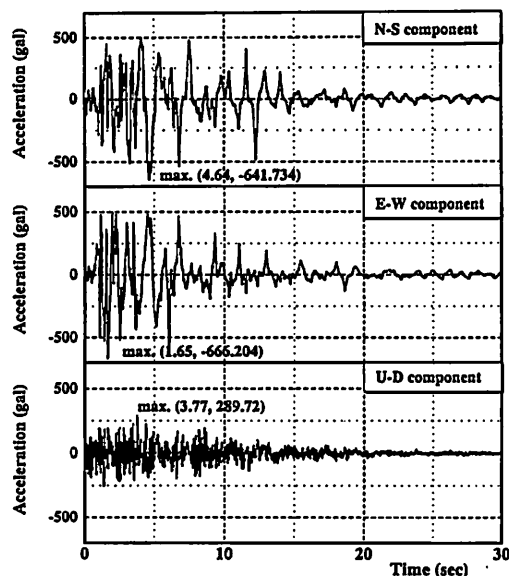


Fig. 3 Strong ground motion recorded at JR Takatori observatory

acting on the steel tower from the stiffening girder is neglected. For the numerical analysis, the geometry and the structural properties of the steel tower is shown in Fig. 1, the tower structure has rectangular hollow steel section with internal stiffeners, which has different dimensions along the tower height and its horizontal beam as shown in Table 1.

(2) Damping mechanism

The damping mechanism in cable-stayed bridges is not well understood. A correct representation of structural damping cannot be formulated from a practical standpoint. Accordingly, simplified assumptions have to be used. In this analysis, the modal damping is treated in two

categories. First, considering physical elastic-plastic hysteresis loss in energy dissipation systems uses a phenomenological damping approach. In the second category, the damping matrix must be explicitly evaluated, where an equivalent viscous damping is introduced in the system in the form of damping matrix [C], as shown in Fig. 2.

In this study, two spectral damping schemes are used: The first scheme based on the matrix of simplified damping assumption is used only to clarify vertical ground motion and damping scheme effects on the steel tower dynamic response, where the attenuation of tower structure is adopted the viscous damping of mass proportional type with damping coefficient to the first fundamental natural vibration mode as standard 2%. In the second scheme, Rayleigh's damping is used to form damping matrix as a linear combination of mass and stiffness matrices, which effectively captures the tower structures damping and is also computationally efficient. The damping ratio corresponding to the frequencies of the fundamental in-plane and out-plane modes of tower free vibration is set to 2%, this Rayleigh's damping scheme is adopted for all present parametric study.

(3) Selected input ground motions

In the dynamic response analysis, the seismic motion by an inland direct strike type earthquake that was recorded during Hyogoken-Nanbu earthquake 1995 of high intensity but short duration is used as an input ground motion to assure the seismic safety of bridges. The horizontal and the vertical accelerations recorded at the station of JR Takatori observatory^{3), 4)}, as presented in Fig. 3, are suggested for dynamic response analysis of the steel tower of cable-stayed bridge at type II of soil condition due to its capability of securing the required seismic performance during the bridge service life. The selected ground motion has maximum acceleration of its components (N-S, E-W and U-D) equal to 642, 666 and 290 gal, respectively. From Fourier spectrum analysis, the predominant frequencies for N-S and E-W components are 0.83 and 0.81 Hz, respectively, which are relatively low, and that for the U-D component is 7.96Hz, which includes a high frequency components and indicates the vertical motion time lag to horizontal motions.

(4) Vertical ground motion component

Generally the vertical ground motion attenuates more rapidly than the horizontal motion, and the vertical motion effects are more evident in the near earthquake field. In fact it was observed that the vertical to horizontal peak ground acceleration ratio tends to assume greater values in the near field and

to decrease as the epicenter distance increases²²⁾. Moreover, another aspect is not taken in consideration is the frequency content of the vertical motion, which is noticed to be significantly higher than that of the horizontal motion. Therefore the vertical component may be more dangerous as it is retained, since it may be close to the vertical frequencies of free vibration of many structures. There are a lot of records of instrumented structures, especially from the 1994 Northridge and 1995 Kobe earthquakes, which show a great amplification of the vertical ground motion.

(5) Natural vibration analysis

According to a number of full scale tests conducted for cables-stayed bridges, it is well known that the natural frequencies and natural mode shapes can be predicted with acceptable accuracy by means of linear elastic analyses that assume appropriate mass and stiffness distributions²³⁾. Depending on the fundamental frequencies of the steel towers of cable-stayed bridge in relation to the dominant frequency content of the seismic input motion, shifting the natural period T of the tower would significantly reduce acceleration responses and tower member forces. The natural vibration analysis is carried out for the previous described steel tower modal. The natural periods, the effective modal mass and the damping coefficient for different vibration modes obtained from the analysis are listed in Table 2. The lowest vibration mode, with a 2.072 sec period, involves transverse vibration of the entire tower structure (right angle to the bridge axis). The second mode (0.9335 sec) is the longitudinal vibration (bridge axial direction). The vertical vibration modes have period of 0.5235 sec to 0.1559 sec including modes 4 and 8. It is apparent that the contributions for the first and second modes are not only of larger values. But, there is another mode of vibration with large value of contribution factor showing a very complicated dynamic behavior.

Table 2 Summary of principal vibration modes for tower model

Mode order	Period sec.	Effective mass as a fraction of total mass	Viscous damping percent	Mode type
1	2.0723	33.195	2.00	H ₁
2	0.9335	30.330	2.00	L ₁
3	0.7726	0.000	2.18	T ₁
4	0.5235	0.034	2.81	V ₁
5	0.3751	1.735	3.68	L ₂
6	0.3625	0.080	3.79	H ₂
7	0.3296	0.000	4.12	T ₂
8	0.1559	34.079	8.35	V ₂
Sum	--	99.423	--	--

H: transverse vibration (in-plane), T: torsional vibration
L: longitudinal vibration (out-plane), V: vertical vibration

4. EFFECTS OF VERTICAL GROUND MOTION AND DAMPING

The principal effect of the vertical motion is the generation of fluctuating axial forces uncoupled from the lateral forces in tower legs. These axial forces are added to the axial forces correlated with the in-plane moments and as a result the compression and tension axial forces in tower legs could reach greater level than that with the horizontal motions alone. The effects of the vertical motion on tower seismic response are studied with the tower model described before. Two cases of analysis for the input ground motion for each spectral damping scheme (mass proportional or Rayleigh's damping) are considered as follows: The first; under the horizontal components only, and the second; under both the horizontal and the vertical components. The vertical excitation main effects can be stated and briefly explained in the following.

(1) Effects on tower axial response

In order to observe directly some of the effects of the vertical motion, the axial force time histories at the tower base are given in Fig. 4. For the mass proportional damping, it is evident the superposition of the axial force variations induced by the vertical motion with that due to the horizontal motions and a higher frequency feature. The axial force extreme values could reach great values with the vertical motion, since the maximum compression axial force at the tower base with and without consideration of vertical ground motion becomes, in fact, 4.01 and 3.13 times the initial axial force due to gravity loads, while the maximum tension axial force becomes about 2.46 and 1.36 times the initial axial force, respectively. The contribution of the vertical motion to the total axial force can be comparable to that of the horizontal motions, since that contribution to the extreme compression and tension values of axial forces in the tower base increases and it reaches about 28% and 81% of the total axial force for that case without consideration of vertical ground motion, respectively.

For the Rayleigh's damping scheme, it can be concluded that the vertical motion has slightly effects in axial forces generation, which are uncoupled to that due to lateral forces and have a higher frequency. The axial forces at tower base due to the overturning moments are significant and the vertical motion has small contribution to the total axial force compared to that of the horizontal motion, since the extreme compression and tension values of axial forces in the tower base indicate that the contribution of the vertical motion reaches about 10% and 6% of the total axial force for that case without vertical ground motion, respectively. For the

case of horizontal input ground motions only, the tower dynamic response is slightly affected by the damping scheme. Moreover, it can be concluded that the damping scheme in the dynamic analysis essentially affects structure seismic response.

(2) Effects on tower flexural response

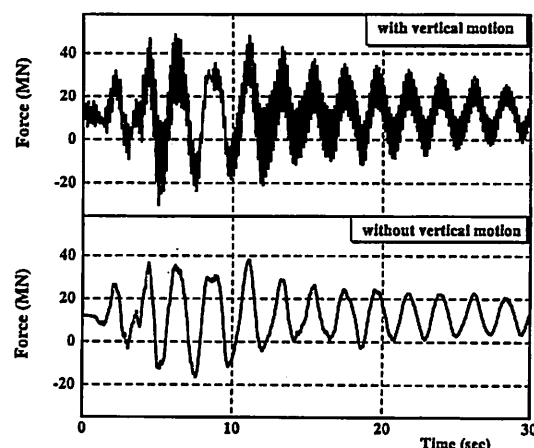
The moment-curvature diagram of the tower base, obtained with and without vertical motion, as shown in Fig. 5, presents the characteristics of the column behavior under coupled axial and lateral force variations. For the mass proportional damping, it can be observed that the asymmetry strength and moment curvature diagram shift due to fluctuating axial force effect. The diagram obtained with the vertical motion reveals an unusual and irregular shape with significant fluctuations in strength and stiffness due to the axial force-moment interaction. The contribution of vertical motion in maximum curvatures reaches about 31% of that case without vertical motion, and this indicates the lower dissipation capacity considered with the vertical excitation. However this is not only reason of the lower dissipated energy, since the tower hysteresis energy decreases as the curvature experiences greater values, moreover another cause is the irregular shape of the hysteresis loops. It can be seen

also the growth of the inelastic behavior with the vertical ground motion, since the axial force fluctuation has the ability of generation of new plastic zones through the tower elements.

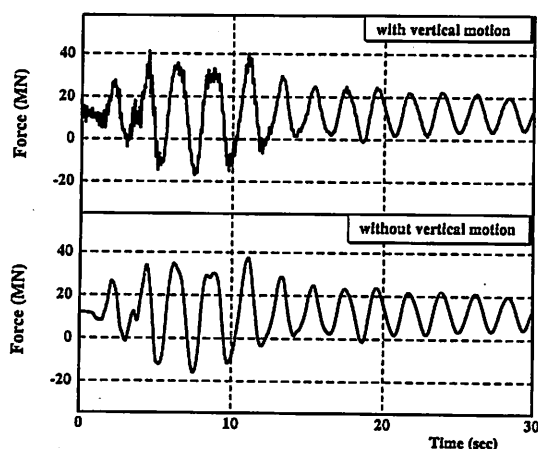
For the Rayleigh's damping scheme, the tower response with vertical ground motion shows very slightly effects in the tower flexural behavior. Moreover, The damage caused by the horizontal motions is similar to that caused by the vertical and horizontal motions. Since tower overall flexural response is not significantly altered by the fluctuation in the axial force associated to the vertical excitation. It can be observed that the analysis using Rayleigh's damping scheme displays more growth of tower inelastic behavior that can be attributed to greater input energy of ground motion leading to cause more damage and ductility demand.

(3) Effects on tower acceleration response

The vertical motion detrimental influence on tower acceleration response can be clarified through acceleration time history study. For the first damping scheme, as shown in Fig. 6(a), the tower top horizontal acceleration is resulted to increase of a non-negligible amount with the vertical motion. The extreme in-plane acceleration values are seen that the contribution of the vertical motion increases and it reaches about 81% of that for case without consideration of vertical ground motion, the in-plane acceleration for the cases with and without considering the vertical motion could reach 5.94 and 3.28 times that of the input ground motion, respectively. For the second damping scheme, it can be concluded that the vertical motion has no significant effect on the acceleration tower response, but it is characterized by effectiveness response and greater energy that causes more damage for tower

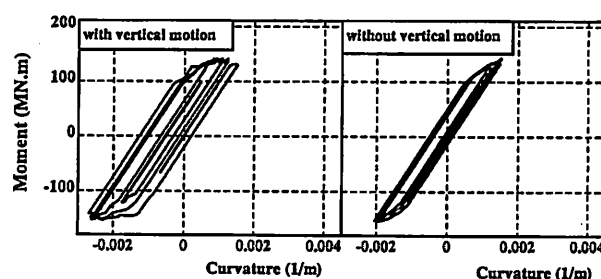


(a) Mass proportional damping scheme

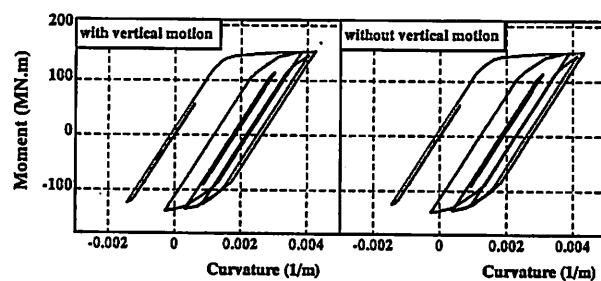


(b) Rayleigh's damping scheme

Fig. 4 Vertical force time history at tower base

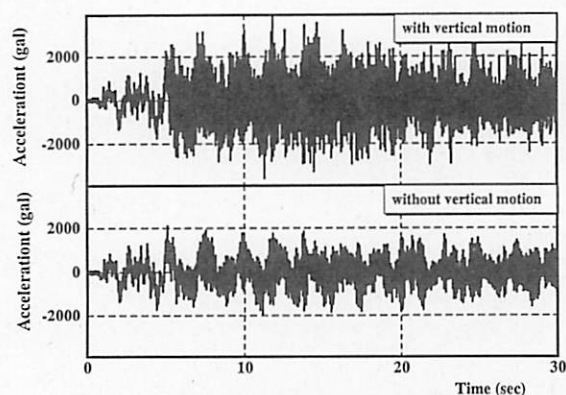


(a) Mass proportional damping scheme

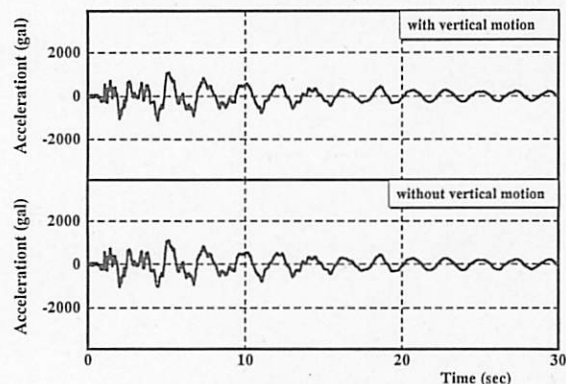


(b) Rayleigh's damping scheme

Fig. 5 Moment-curvature diagrams at tower base



(a) Mass proportional damping scheme



(b) Rayleigh's damping scheme

Fig. 6 In-plane acceleration time history of the tower top

structure, as shown in Fig. 6(b). The vertical ground motion effect on seismic response depends totally on the spectral damping scheme considered for nonlinear dynamic analysis. For the mass proportional damping scheme, the vertical motion displays significantly effect on tower seismic response, which may be attributed to the amplification of high frequencies mode of tower vibration included in the frequency content of the vertical motion, in terms lead to overestimation of the tower response. For the Rayleigh's damping scheme, it is appeared that the vertical motion has slightly effect on tower dynamic behavior due to high damping ratio for high frequency modes, which is pronouncedly affected by the vertical ground motion that has high frequency content compared with that of horizontal ground motions. The Rayleigh's damping could be recommended for conservative nonlinear seismic response of high-rise tower structures.

5. EFFECTS OF CONSTRUCTION INITIAL IMPERFECTIONS

(1) Initial geometric imperfections

Initial imperfections in structures that have not been subjected to damage usually result from the fabrication process and the strength of a thin-walled

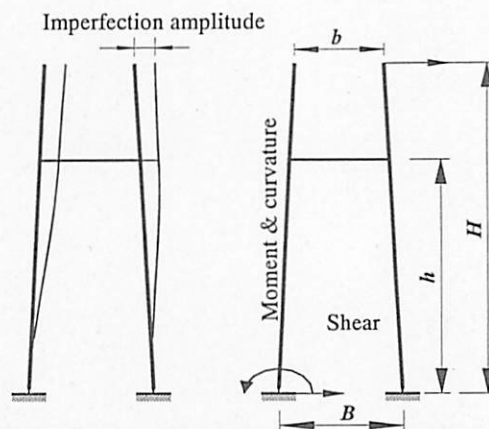


Fig. 7 Initial geometric imperfection pattern and different measured aspects of tower response

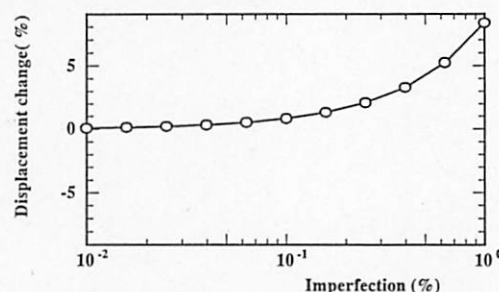


Fig. 8 Displacement & imperfection amplitude relationship

steel member is particularly sensitive to imperfections in the shape of its natural vibration modes. The amplitude of imperfections in the lowest eigenmodes of vibration is often sufficient to characterize the influential imperfections¹⁹⁾. The initial geometric imperfection is applied by modifying the nodal coordinates using a field created by scaling the appropriate vibration eigenvector obtained from an elastic natural vibration analysis of tower model.

The dynamic response of the steel tower with consideration of initial geometric imperfection is investigated for different imperfection amplitudes of range from 0.01 up to 1% of tower height. The initial geometric imperfection is taken to be the first fundamental vibration mode pattern. Fig. 7 describes the mode shape of the steel tower obtained from eigenvalue analysis. The mode is normalized so that the modal displacement at tower top is adjusted to the imperfection amplitude. The effects of the magnitude of initial geometric on the extreme values of in-plane displacement of tower top, shear force, in-plane moment and curvature at tower base are presented.

It can be concluded that the initial geometric imperfection within its design range of upper limit equal to 0.1% slightly affect on the tower seismic response but beyond this range, these effects are characterized by rapidly and nonlinearly increase and become significant as the initial imperfection approach severe values (1%). The extreme values of

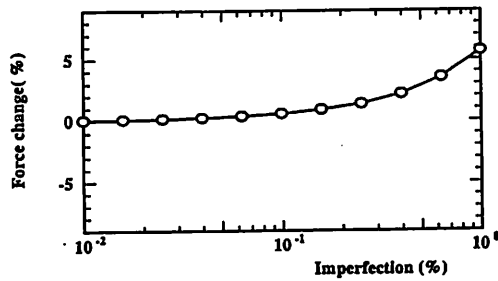


Fig. 9 Shear force & imperfection amplitude relationship

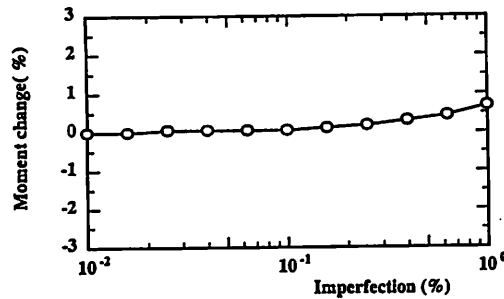


Fig. 10 Moment & imperfection amplitude relationship

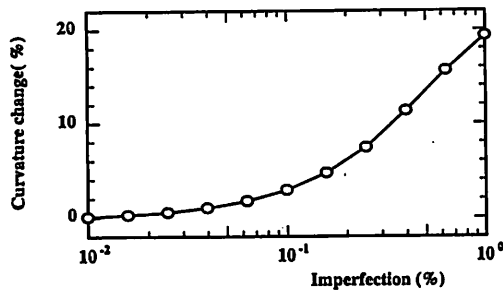


Fig. 11 Curvature & imperfection amplitude relationship

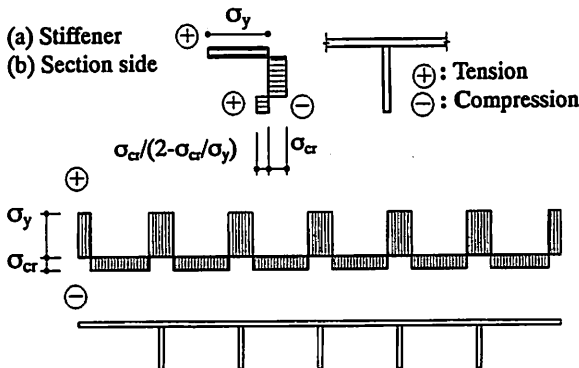
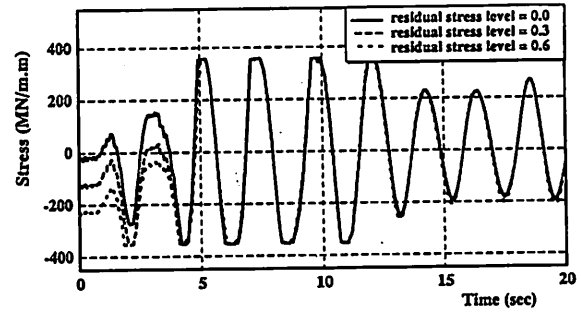
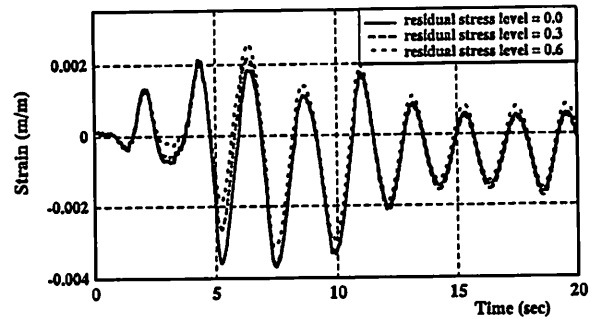


Fig. 12 Residual stresses in each side tower cross section

in-displacement at tower top and shear at tower base increase as imperfection amplitude increase under the same loading and have values 8% and 6% of that of perfect tower at maximum imperfection 1% of tower height as given in Figs. 8 and 9, respectively. But the extreme values of bending moment at tower base slightly increase up to 1.5% of that of perfect tower as a result of tower inelastic response that is characterized by large deformation corresponding to small force response, as illustrated in Fig. 10. Moreover, the imperfection amplitude has pronounced effects on curvature at tower base and



(a) Stress of compression residual stress fiber at tower base



(b) Stain of compression residual stress fiber at tower base

Fig. 13 Stress and stain time histories

as a consequence the tower structure response, since the curvature increases up to 20% of that of perfect tower as shown in Fig. 11.

(2) Residual stresses

The presence of longitudinal residual stresses in stiffened rectangular hollow cross section is mainly attributed to the welding of stiffening members in addition to hot rolling of the hollow cross section. The residual stresses in the weld, stiffener and hollow section material in the vicinity of the weld are close to the yield as a result of the contraction of the welds. The magnitude and distribution of the residual stress are governed by welding parameters such as heat input and cooling rate of the welding process adopted for fabrication processes. To model the distribution of longitudinal membrane residual stresses in the cross section and the plasticity spread, the integration through the division of fiber model is considered to be sufficient. Fig. 12 illustrates a typical residual stress pattern²⁴⁾ of tower cross section that is used in the computational model. It has been presented that the magnitude of the compressive residual stresses increases as the component plates of hollow section slenderness becomes smaller, values of stress up to 75% of the yield strength have been measured²⁵⁾.

The formation of residual stresses is inevitable in any welding and hot rolled operations, when they are superimposed on the externally applied stress fields, the residual stresses can result in significant differences in the performance of welded structures. The effects of residual stresses on the dynamic

response of the steel tower is studied by varying residual stress level, where the compressive residual stresses range from 0.0 up to 0.60 times with interval of 0.1 of the yield stress are used to represent the condition of stress relieved, from lightly welded to heavily welded stiffener to the cross section. It can be observed that residual stresses have a more pronounced effect on the tower dynamic response, which will yield at a lower load stress due to the presence of compressive residual stresses. It can be seen how the applied membrane stresses is used to reflect the removal of residual stresses through the first cycle of the stress response in Fig. 13 (a). But the residual effects extent over most of time history of strain response, and could be presented as peak time lag and decreasing plastic deformation capability as illustrated in Fig. 13(b). The tower top displacement decreases as the residual stress level increases, due to plastic deformation decrease as shown in Fig. 14.

In addition, the tower strength and stiffness are reduced by the presence of residual stresses and decrease with residual stress level increase as seen in Figs. 15 and 16. The tower flexural response is

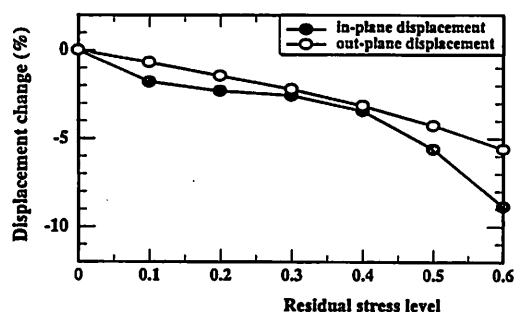


Fig.14 Displacement & residual stress level at tower top

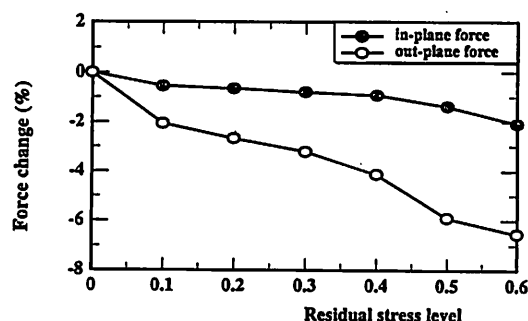


Fig. 15 Shear force & residual stress level at tower base

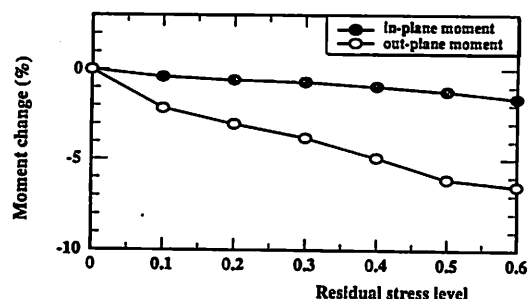
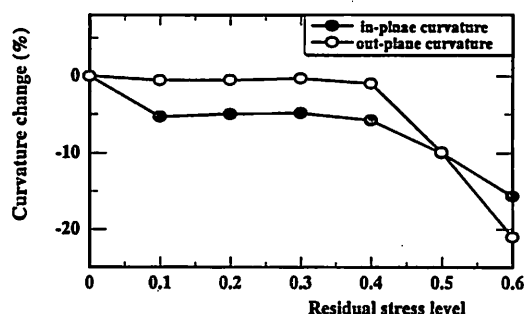


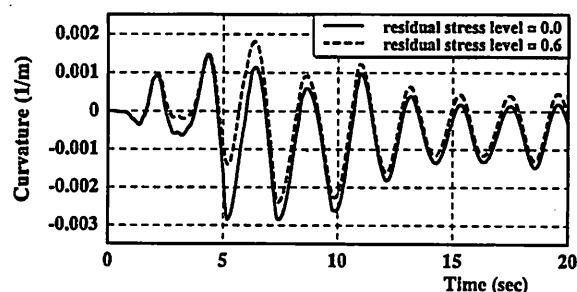
Fig. 16 Moment & residual stress level at tower base

slightly affected by the presence of residual stress up residual stress level about 0.4, behind this level, the tower base curvature abruptly increases up to 20% of that of the original tower, as illustrated in Fig. 17(a), this behavior could be attributed to the peak time lag in curvature time history response as indicated in Fig. 17(b). From the total, damping and strain energies of the whole tower study, it is appeared the energies decrease in gradually as the compression residual stress increases and much effect on strain energy. This reduction can be attributed to decreasing of tower plastic deformation capability and gradual decreasing of tower stiffness due to presence of compression residual stresses, as given in Fig. 18.

In general, it can be concluded that the residual stress effects on the tower seismic response are sensitive to its stiffness in the longitudinal and transverse directions. Since the tower deformations response in the out-plane direction are more sensitive to the existence of residual stresses, which can be attributed to the long period of vibration and more flexibility of tower in-plane direction compared



(a) Curvature & residual stress level at tower base



(b) Time history

Fig. 17 Curvature at tower base

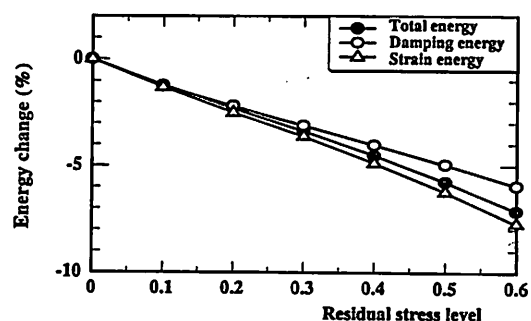


Fig. 18 Total, damping and strain energies of the whole tower & residual stress level relationship

to that of the out-plane direction. The residual stresses within the design range of upper limit equal to 0.3 affect the tower seismic response not more than 3% and 5% for displacement and in-plane curvature demand, respectively. Also, these effects are not more 4% for both out plane shear and flexural capacity. The other aspects of tower seismic response are slightly affected by residual stresses values up to the design range upper limit. The residual stresses have the time lag effect on tower peak response, which can be noted from curvature rate of change at residual stress level of 0.4.

6. LOW YIELD MATERIAL ENERGY DISSIPATION SYSTEM

The design of a passive energy dissipation system depends on many factors, including the period of original tower configuration, the input ground motion response spectrum, force deformation relationship, and so on. The design of isolation system should be able to provide supplemental damping to significantly reduce tower response to ground motion and dissipation a large portion of earthquake input energy through inelastic deformations in certain positions, which could be easily retrofit after damage. Thus the spectral acceleration and structural element forces could be significantly reduced when they are compared to that of original tower. The nonlinear dynamic behavior and seismic performance of the steel tower under three dimensional great earthquake motion are studied for three different cases, a case of original tower and other two cases of proposed energy dissipation system. In these two cases, the concentration of inelastic behavior along the tower horizontal beam by reduction of its strength is considered, this reduction is done by using low yield steel material instead of cross section dimensions reduction. Two yield stresses are used that equal to 235 MPa and 100 MPa corresponding to medium and low yield level equal to 0.67 and 0.28 relative to that of original tower, respectively.

The performance of the proposed energy dissipation system is analyzed by comparing the energies time history, where the input energy is defined as the total energy related to inertia forces induced by the ground motion. It is appeared that the proposed energy dissipation system has two effects on the tower response: The first is to increase the tower structural system ability to reflect a portion of earthquake input energy, since as the horizontal beam yield early attains, the tower becomes more flexible. In effect, the increased flexibility acts as a filter. Secondly, it increases the amount of damping and dissipation energies through

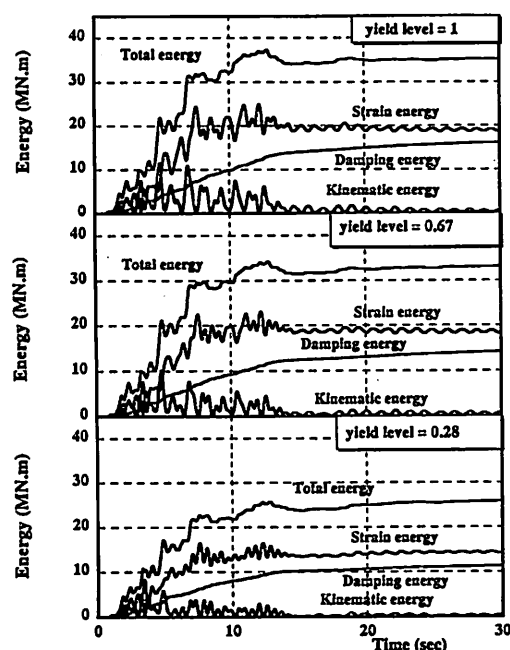
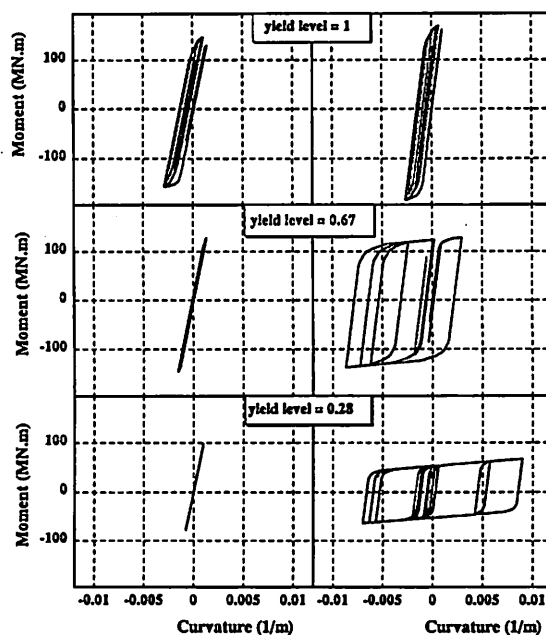


Fig. 19 Different energies time history of the whole tower



(a) At tower base (b) At horizontal beam end

Fig. 20 Moment & curvature relationship

inelastic deformation hysteresis. The calculated results of different yield levels show effective energy dissipation through the horizontal beam. As the yield level decreases, the energy dissipation system becomes more effective in energy absorption and damage control, as seen in Fig. 19.

As the yield level of horizontal beam material decreases, thus the load capacity of tower horizontal beam decreases and forces redistribution in tower structural elements occurs. It can be seen from Fig. 20(b) that more concentration of inelastic behavior and ductility at tower horizontal beam is attained as yield level change for low values, which

is easy to inspect and repair if necessary. The rest of the structure approaches elastic behavior as yield level decreases, thus there is a possibility of eliminating permanent damage and minimizing the extent of retrofit. The main tower parts attain almost elastic behavior at study case of yield level equal to 0.28, as seen in Fig. 20(a).

In general, the isolated tower exhibits elastic response due to the redistribution of the seismic forces to the tower elements in accordance to their strength. It can be concluded that the proposed energy dissipation system is effective in controlling the maximum tower displacement, since the displacement tower response decreases as the horizontal beam capacity decrease, as illustrated in Fig. 21. The better performance of the isolation proposed energy dissipation system is indicated by comparing the reaction force time history at the tower base for different levels of yield strength of the horizontal beam. It is illustrated from Figs. 22 and 23 that the isolated tower provides pronounced reduction in the reaction forces response compared to the original tower response, this reduction becomes more pronounced as the yield level decreases, due to seismic forces redistribution to tower elements according to their strength and stiffness. The proposed energy dissipation system effect can be understood from the in-plane shear and vertical force time histories at tower base. The tower main structures approach elastic behavior as yield capacity of horizontal beam decreases. Since for yield level equal to 0.67, tower flexural response represented in the moment ratio relative to yield moment capacity (M_y that equal to 130 MN·m) time history at tower base, enters the plastic response four times compared with the original tower enters

the plastic response seventh times, while for low yield level equal to 0.28, the elastic behavior is displayed along the time history, as seen in Fig. 24.

The hysteretic dampers of low yield steel material can provide relatively large energy dissipation through their materials that are strained beyond their yield limits, thus can be cost effective. The hysteretic dampers cannot be activated as dampers unless their materials receive inelastic excursions, so the hysteretic dampers are effective only for larger earthquake excitation that can be understood from this study, but fail in providing the required damping for smaller vibrations. Another aspects of post-yield buckling capacity should be considered in the design. In this case of study, as horizontal beam receive inelastic excursions; the buckling effective length could get larger, as a result, the load carrying capacity decreases.

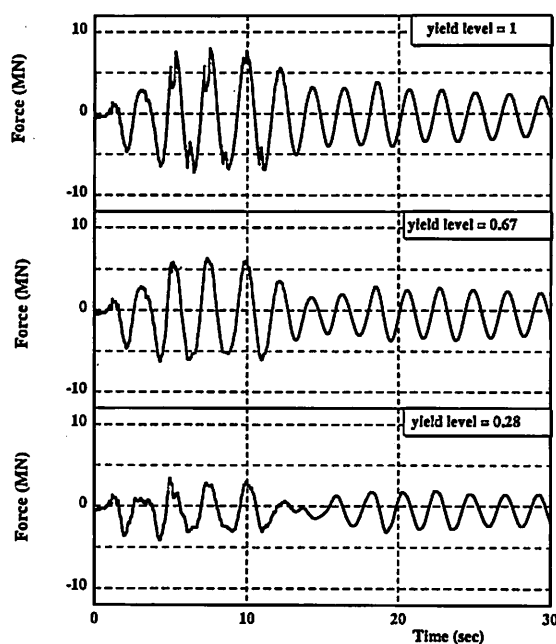


Fig. 22 In-plane shear force time history at tower base

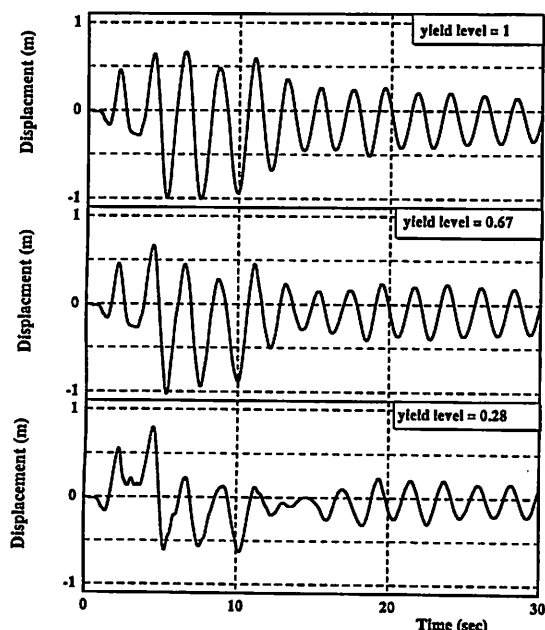


Fig. 21 In-plane displacement time history at tower top

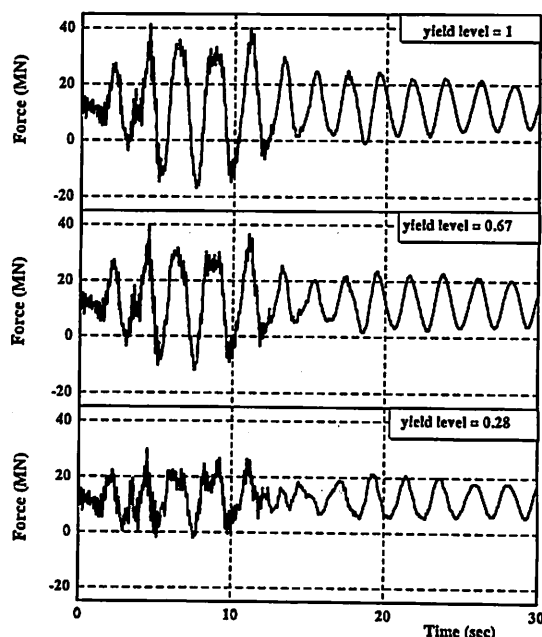


Fig. 23 Vertical force time history at tower base

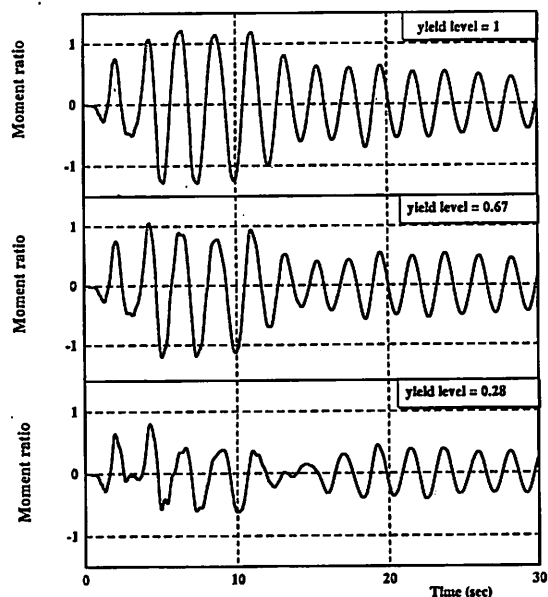


Fig. 24 Moment ratio time history at tower base

The buckling load carrying capacity is calculated for the worst case of no contribution of horizontal beam to the tower legs stiffening, it is found to be equal to 49.5 MN, which is much greater than the corresponding tower response. Moreover, the proposed low yield hysteretic dampers lead to effective reduction of vertical force response as yield level decreases, as a result buckling demand decreases, as shown in Fig. 23.

7. EFFECTS OF TOWER MODAL SHAPES

The natural vibration periods of the in-plane, out-plane and torsion fundamental modes of the steel tower with different modal shapes have been studied for different height and length of the horizontal beam. A detailed natural vibration characteristic is provided for horizontal beam different height relative to the total tower height (height ratio, $h/H = 39/68 - 68/68$) and different width at tower top relative to that at tower base (length ratio, $b/B = 13/18 - 3/18$). Fig. 25 presents the natural period of these three modes of vibration and height and length ratios relationship.

The effectiveness of the tower modal shapes can be measured by their capabilities in the energy dissipation through shifting natural period of the fundamental mode and increasing structural damping. The natural vibration analysis indicates that the height and length of tower horizontal beam have effective role in shifting the primary period of the tower free vibration, and slightly affect the higher mode of vibration. Longer natural period can be attained by either horizontal beam height (h/H) relative to tower height or tower top to base width ratio (b/B) increase. For the original steel tower of cable-stayed

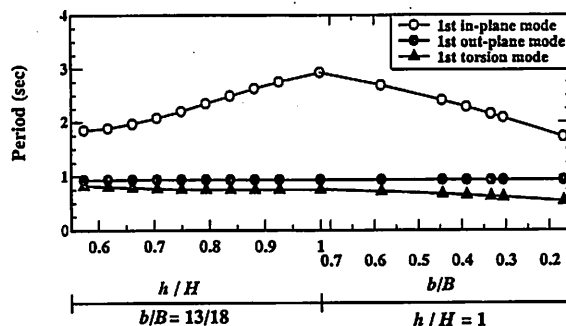


Fig. 25 Natural period & horizontal beam height/length relation

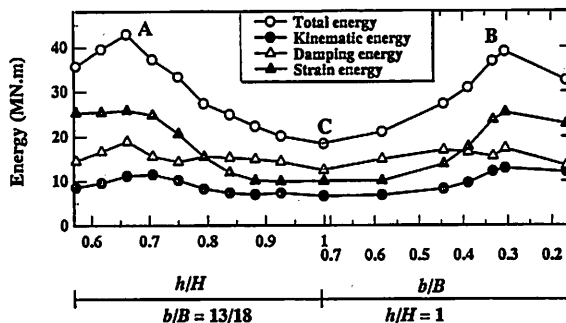


Fig. 26 Total, damping and strain energies of the whole tower & horizontal beam height/length relationship

bridge, the better performance could be attained by moving the horizontal beam from its current designed at $h/H = 48/68$ toward tower top. This modification in tower design could lead to natural period about 1.5 times longer, hence reduces the seismic demand required for earthquake mitigation.

It is indicated from Fig. 26 through the energies extreme values study that the horizontal beam has two critical positions corresponding to point A ($h/H=45/68$, $b/B=13/18$) and point B ($h/H=68/68$, $b/B=5.5/18$), where the maximum total energy occurs, as a result the seismic demands are increased. The kinematic and damping energies is slightly affected by the horizontal beam position of tower, while the total and strain energies along the range between critical points A and B decrease highly nonlinear as either the height ratio or the length ratio increase up to optimum position of horizontal beam corresponding to point C. The tower base shear and vertical force variations with different horizontal beam height and length ratios are described in Figs. 27 and 28, respectively, which confirm the optimum position of horizontal beam at tower top as detected from the energetic study. The height of horizontal beam has effective role in the dynamic response of steel tower depends to large extent to tower shapes¹⁵, including the horizontal beam position and strength represented in material yield level. The effects of height and length of the horizontal beam on tower dynamic response are discussed through time history analysis to examine the tower shapes effects. For this purpose, three different cases of tower modal shape, as illustrated in Fig. 29, are considered as follow:

- Case I: The original tower of cable-stayed bridge.
- Case II: Portal frame-type where the horizontal beam in case I is raised to tower top.
- Case III: Inverted V-type where the tower top length decreases from case II up to 3 m.

The natural period of the predominant modes of free vibration for studied cases of tower model shapes is given in Table 3, where the symbols H_1 , L_1 and T_1 are the first natural mode in the direction of the right angle to bridge axis (in-plane), of bridge axis (out-plane) and torsion, respectively. The first natural periods of out-plane and torsion modes are smaller than that of in-plane mode due to the stiffening of cables. It is seen that the natural period

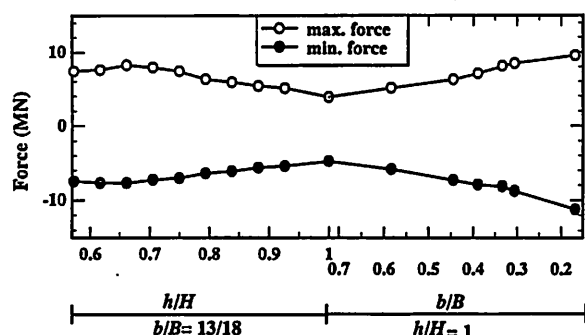


Fig. 27 In-plane shear at tower base & horizontal beam height/length relationship

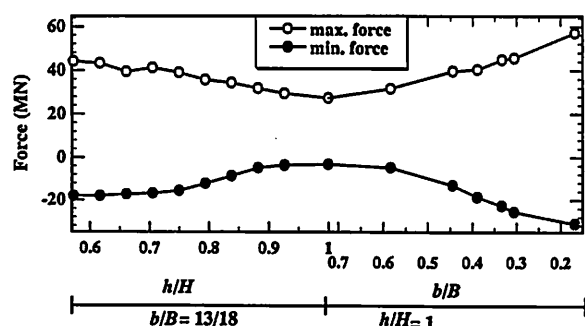


Fig. 28 Vertical force at tower base & horizontal beam height/length relationship

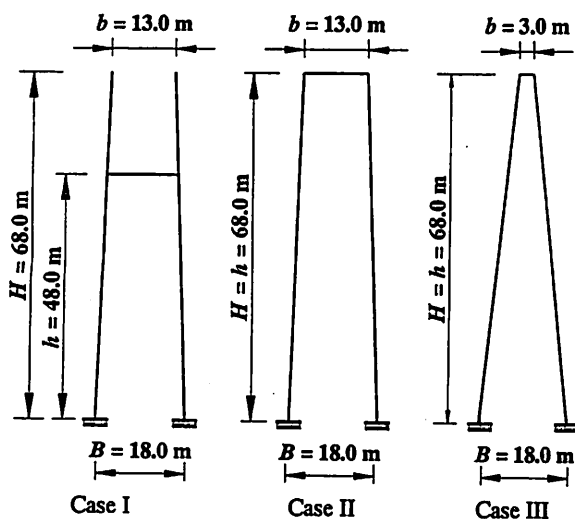


Fig. 29 Tower modal shapes

of the portal frame-type tower has tendency to be getting longer in compared with the inverted V-type and original tower (H-type) because of more flexibility of tower frame structure due to its height increases and also there is a massive horizontal beam situated on the tower top.

Fig. 30 displays the energy time history for different cases of tower modal shape. The portal frame-type tower has enough flexibility to reflect most of the absolute input energy by flexibility filtering; the damping energy becomes larger and reaches about 80% relative to input energy. Moreover, the strain energy dissipated through tower inelastic hysteretic deformation decreases, since the tower still keeps elastic behavior as illustrated in Fig. 31. The hysteresis of the other types of tower modal shape (cases I and III) becomes larger and pronouncedly displays inelastic behavior especially for H-type. The largest inelastic deformation damage has been attained, leading to high strain energy dissipated through tower hysteric behavior. Moreover, the total input energy is getting higher, which requires greater seismic demand for mitigation of earthquake hazards. Fig. 32 indicates the trajectory of tower top displacement response in the two horizontal directions. The right tower top

Table 3 Natural period for tower different modal shapes (sec)

Mode type	Cases of tower modal shapes		
	Case I	Case II	Case III
H_1	2.072	2.928	1.727
L_1	0.934	0.935	0.935
T_1	0.773	0.761	0.543

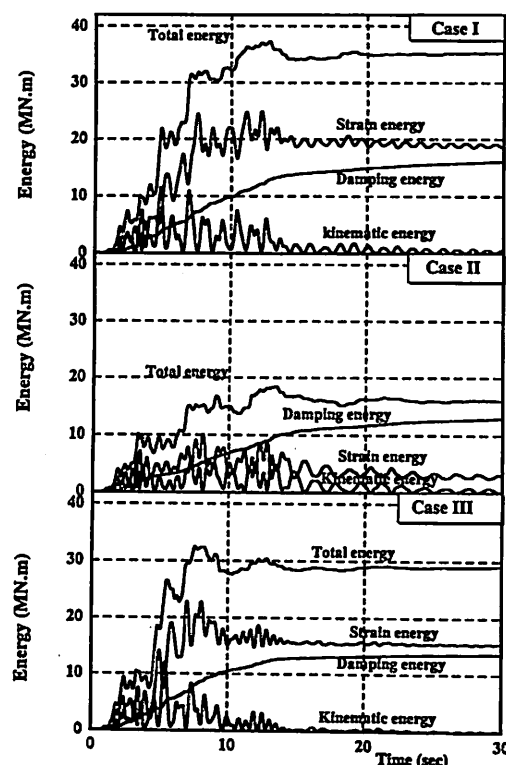


Fig. 30 Different energies time history of the whole tower

displacement response is considered. The vertical axis is the in-plane displacement (right angle to bridge axis direction) and the horizontal axis is the out-plane displacement (bridge axis direction). The results indicate that all studied three cases are characterized by in-plane larger displacement than that of the out-plane response, which is related to long period of the in plane free vibration compared with the out-plane vibration. In addition, the existence of cables reduces tower shape effects on the out plane tower displacement response. The H-type (case I) tower has the largest in-plane displacement response among three considered tower shapes due to the absence of frame action at tower top. The tower part above the horizontal beam approaches to a cantilever behavior, also it is noted the asymmetric in-plane displacement that related to torsional vibration effects. The portal frame-type (case II) tower displays reasonably large displacement, since it has the longest period of vibration and massive effect of the horizontal beam

at tower top. The inverted V-type (case III) tower model displays the smallest top displacement response in both directions due to stiff frame action of this shape and short period of in-plane and torsional vibrations. The dynamic response of each tower model is examined on axial force at tower basement as illustrated in Fig. 33, which shows the orbits of axial force response that occurs at both sides of the tower basement.

The vertical and horizontal axes are the axial force response at right and left sides of tower basement, respectively. A dead load equivalent to the stiffening girder weight is equal to 12.15 MN and it acts on both sides of the tower basement through cables. It is noted that tower modals have symmetrical form of axial force dynamic response around the dead load. The inverted V-type has the largest axial force response, where the maximum compressive and tension axial forces are about 4.70 and 2.55 times of the gravity loads, the large negative reaction makes the problem that the anchor

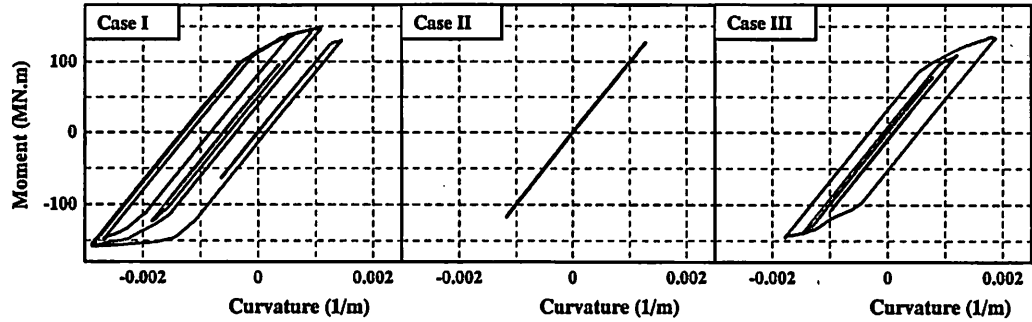


Fig. 31 Moment curvature relations at tower base

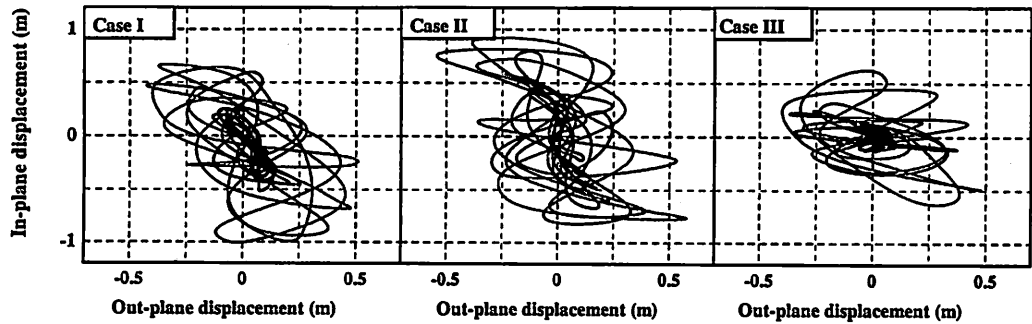


Fig. 32 Trajectory of response sway displacement of tower top

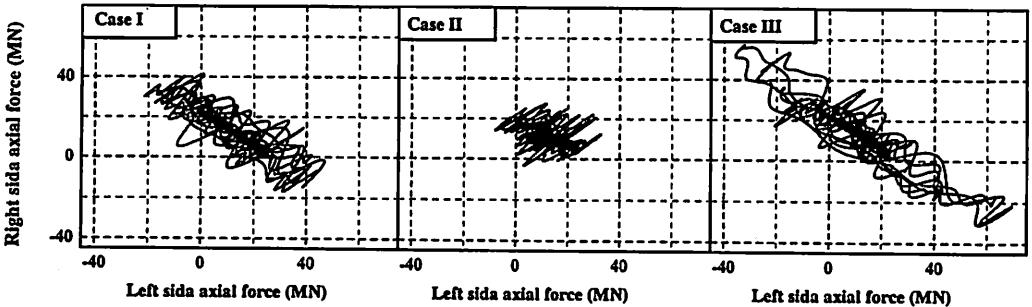


Fig. 33 Axial force relation at tower bases

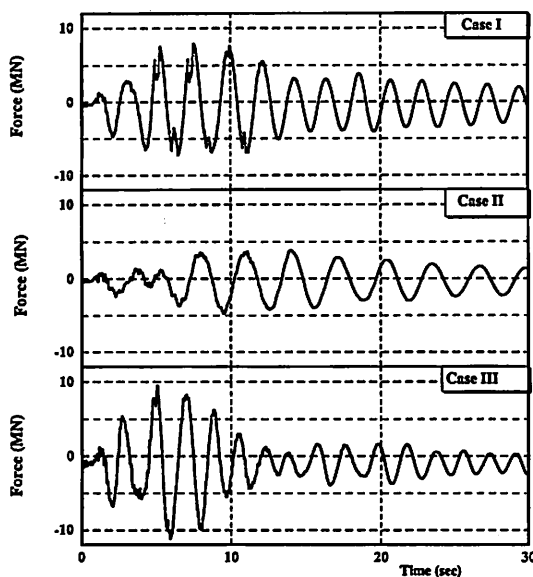


Fig. 34 In-plane force time history at tower base

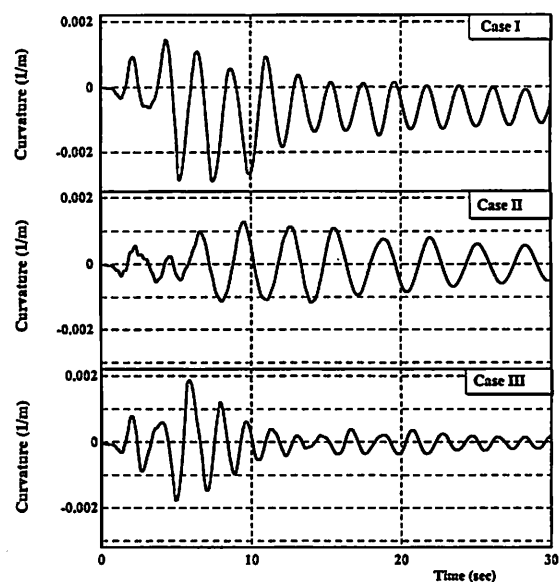


Fig. 36 In-plane curvature time history at tower base

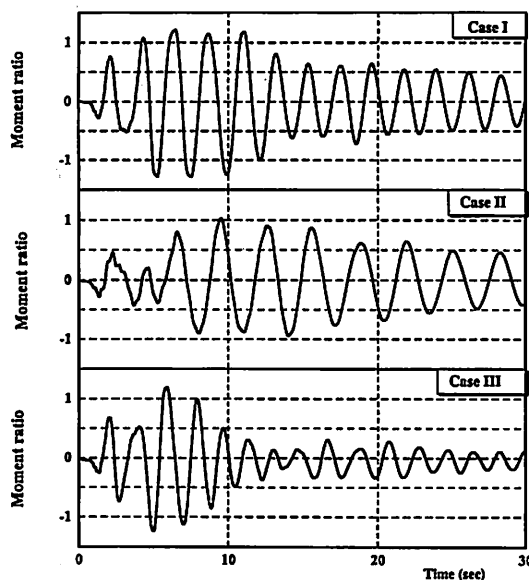


Fig. 35 In-plane moment time history at tower base

bolts may fail in the lift up and the safety of the tower should be considered. The same behavior but slightly less severe can be seen for the H-type (original tower), where the maximum compressive and tension axial forces are about 3.39 and 1.39 times of the gravity loads. On the other side, the portal frame-type (case II) model displays reasonable axial force response with small amplitude, since the maximum compressive and tension axial forces are about 2.26 and 0.27 times of gravity loads. The current results prove again the effectiveness of the horizontal beam position on tower axial force dynamic response. The change of horizontal beam height from that of the original tower ($h/H=48/68$ m) toward tower top ($h/H=68/68$) will reduce the compressive and tensile axial forces at tower base about 34% and 81% of that of original tower, respectively, and almost the uplift problem disappears that secure the tower seismic response

against anchor bolts lift up failure. The severe axial force response of the inverted V-type could be attributed to inclination of the tower legs that leads to significant flexural-axial interaction and increases the flexural contribution in the tower vertical force response under external seismic excitation.

The reaction force time history at the tower base for different tower shapes could indicate the better performance of the tower modal shapes, as indicated in Fig. 34. It is found that the portal frame-type tower (case II) provides pronounced reduction in the reaction force response compared to the original tower response (case I), and its response is characterized by long period of vibration and elastic behavior, while the inverted V-type displays larger shear as well as axial response. From the moment ratio relative to the yield moment capacity (M_y) at the tower base, which is equal to 130 MN·m, the original tower enters the inelastic response many times within the first twelve seconds, which explains the large damage occur.

The inverted V-type tower enters the inelastic response three times within the first ten seconds leading to less damage compared to case I, while the portal frame-type tower still has elastic response along the time history, as shown in Fig. 35. The curvature time history at tower base displays residual deformation and damage for original tower, however, the residual deformations disappear for both portal frame and inverted V-type models due to elastic behavior of portal frame-type and ability of inverted V-type to restore symmetric behavior with no residual deformation even it has inelastic deformation at early stage of response, as illustrated in Fig. 36.

8. CONCLUSIONS

Analytical parametric study on steel tower of cable-stayed bridges is performed for investigation the individual influence of different design aspects on its dynamic characteristics, such as damping scheme, input ground motion, allowable initial imperfections, energy dissipation and tower modal shapes. A finite element procedure based on total Lagrangian formulation for the nonlinear dynamic analysis of steel tower of cable-stayed bridge under three dimensional great earthquake ground motion is carried out. From the performed investigations and discussions, the following conclusions can be summarized:

- (1) The effect of vertical ground motion has highly dependence on damping scheme. The Rayleigh's damping is more effective in high frequency range, which leads to slightly effects of the vertical ground motion on tower dynamic response. The Rayleigh's damping could be recommended for conservative nonlinear seismic response of high-rise towers.
- (2) For mass proportional damping scheme, a significant axial force fluctuations due to vertical motion inertia forces affects the tower behavior and as a consequence the global structural response. The contribution of the vertical motion to extreme compression and tension axial forces at tower base reaches about 28% and 81% of that of horizontal motions only, respectively. Moreover the tower axial forces response is characterized by high frequency. The consideration of horizontal excitations only could underestimate the curvature ductility demand.
- (3) For Rayleigh's damping scheme, the tower response with vertical motion shows slightly effects on its flexural behavior, but more growth of inelastic behavior arises due to greater input energy. The contribution of the vertical motion to extreme compression and tension axial forces at tower base reaches about 6% and 10% of that case of horizontal motions only, respectively.
- (4) The initial geometric imperfections of tower fundamental vibration mode pattern, within its design range of upper limit equal to 0.1%, slightly affect the tower seismic response. But beyond this range, these effects are characterized by rapidly and nonlinearly increase and become significant as the initial imperfections approach severe value (1%).
- (5) The normal stress distribution is affected by the residual stress; as a result, the tower load carrying capacities and stiffness decrease as the residual stress level increases. Moreover, the residual stress has detrimental effects on tower structural performance, which can be characterized by decreasing plastic deformation capability.

(6) The residual stress effects on the tower seismic response are sensitive to its stiffness in the longitudinal and transverse directions. These effects within the design range of upper limit equal to 0.3 on the tower seismic response are not more than 5% that of perfect tower response.

(7) The proposed energy dissipation system demonstrates its capability and effectiveness in reducing structural elements forces and controlling tower maximum displacement through its capability achieving the concentration of inelastic behavior at tower horizontal beam and keeping the rest of the structure elastic behavior as yield level decreases. The post-yield buckling capacity should be considered in the design of tower structures.

(8) This technique could add damping primarily by material hysteresis and increase tower flexibility as the horizontal beam yield early attains, in terms tower structural system ability to reflect a portion of earthquake input energy increases. The energy dissipation system becomes more effective in energy absorption through the horizontal beam.

(9) The portal frame-type tower has the longest natural period and enough flexibility to reflect most of the absolute earthquake input energy by flexibility filtering. The damping energy could reach larger values relative to input energy about 80% and minimum strain energy is dissipated through inelastic hysteric deformations of the tower.

(10) The inverted V-type has the largest axial force response, where the maximum compressive and tension axial forces are about 4.70 and 2.55 times of the gravity loads, respectively. The large negative reaction makes the problem that the anchor bolts may fail in the lift up and the safety of the tower should be considered.

(11) The effectiveness of the horizontal beam position on tower axial force dynamic response is assured. The change of horizontal beam height from that of the original tower ($h/H=48/68$) toward tower top ($h/H=68/68$) reduces the compressive and tensile axial forces at tower base about 34% and 81% of that of original tower and almost the uplift disappears.

REFERENCES

- 1) Ali, H. M. and Abdel-Ghaffar, A. M.: Modeling the nonlinear seismic behavior of cable-stayed bridges with passive control bearings, *Computers & Structures*, Vol. 54, No. 3, pp. 461-492, 1995.
- 2) Hayashikawa, T. and Abdel Raheem, S. E.: Nonlinear dynamic behavior of steel tower of cable-stayed bridges with passive energy dissipation system, *Journal of Structural Engineering, JSCE*, Vol. 48A, pp. 863-874, 2002.
- 3) Committee of Earthquake Engineering: *The 1995 Hyogoken-Nambu Earthquake, Investigation into Damage to Civil Engineering Structures, JSCE*, 1996.

- 4) Japan Road Association: *Specification for Highway Bridges-Part V Seismic Design*, Maruzen, Tokyo, 1996.
- 5) Bozorgnia, Y., Niazi, M. and Cambell, K. W.: Characteristics of free-field vertical ground motion during the Northridge earthquake, *Journal of Earthquake Spectra*, Vol.11, No. 4, pp. 515-525, 1995.
- 6) Papazoglou, A. J. and Elnashai, A. S.: Analytical and field evidence of the damaging effect of vertical earthquake ground motion, *Earthquake Engineering and Structural Dynamics*, Vol. 25, pp. 1109-1137, 1996.
- 7) Diotallevi, P. P. and Landi, L.: Effect of the axial force and of the vertical ground motion component on the seismic response of R/C frames, *Proceedings of 12th. World Conference on Earthquake Engineering*, Paper No. 1026, 2000.
- 8) Saadeghvaziri, M. A. and Foutch, D. A.: Behavior of RC columns under non-proportionally varying axial loads, *Journal of Structural Engineering, ASCE*, Vol. 116, No. 7, pp. 1835-1856, 1990.
- 9) Hayashikawa, T., Aly, G. A., Abdel Raheem, S. E.: Effect of vertical ground motion on seismic response of steel tower of cable-stayed bridges, *Proceedings of Hokkaido chapter of the Japan Society of Civil Engineers, JSCE*, No. 58(A), pp. 112-115, 2001.
- 10) Usami, T. and Fukumoto, Y.: Local and overall buckling of welded box columns, *Journal of Structural Division, ASCE*, Vol. 108, No. 3, pp. 525-542, 1982.
- 11) Zhang, J. and Dong, P.: Residual stresses in welded moment frames and implications for structural performance, *Journal of Structural Engineering, ASCE*, Vol. 126, No. 3, pp. 306-315, 2000.
- 12) Grondin, G. Y., Elwi, A. E. and Cheng, J. J. R.: Buckling of stiffened plates - a parametric study, *Journal of Constructional Steel Research*, Vol. 50, pp. 151-175, 1999.
- 13) Fujikubo, M. and Yao, T.: Elastic local buckling strength of stiffened plate considering plate/stiffener interaction and welding residual stress, *Marine Structures*, Vol. 12, pp. 543-564, 1999.
- 14) Banno, S., Usami, T. and Mizuno, E.: Cyclic elasto-plastic finite displacement analysis of plates with modified two-surface model, *Proceedings of the Japan Society of Civil Engineers, JSCE*, No. 525/I-33, pp. 1-15, 1995.
- 15) Hayashikawa, T., Matsui, Y. and Kaneko, T.: Nonlinear dynamic behavior and seismic isolation of steel towers of cable-stayed bridges under great earthquake ground motion, *Proceedings of 12th. World Conference on Earthquake Engineering*, Paper No. 0469, 2000.
- 16) Nakashima, M.: Strain-hardening behavior of shear panels made of low yield steel. I: test, *Journal of Structural Engineering, ASCE*, Vol. 121, No. 12, pp. 1742-1749, 1995.
- 17) Nakashima, M., Akazawa, T. and Tsuji, B.: Strain-hardening behavior of shear panels made of low yield steel. II: model, *Journal of Structural Engineering, ASCE*, Vol. 121, No. 12, pp. 1750-1757, 1995.
- 18) Nakashima, M., Iwai, S., Iwata, M., Takeuchi, T., Konomi, S., Akazawa, T. and Saburi, K.: Energy dissipation behavior of shear panels made of low yield steel, *Earthquake Engineering and Structural Dynamics*, Vol. 23, pp. 1299-1313, 1994.
- 19) Schafer, B. W. and Peköz, T.: Computational modeling of cold-formed steel: characterizing geometric imperfections and residual stresses, *Journal of Constructional Steel Research*, Vol. 47, pp. 193-210, 1998.
- 20) Chen, C. N.: Efficient and reliable solutions of static and dynamic nonlinear structural mechanics problems by an integrated numerical approach using DQFEM and direct time integration with accelerated equilibrium iteration schemes, *Applied Mathematical Modelling*, Vol. 24, pp. 637-655, 2000.
- 21) Ernst, J. H.: Der E-modul von seilen unter berucksichtigung des durchhanges, *Der Bauingenieur*, Berlin, Vol. 40, No. 2, 52-55, 1965 (in German).
- 22) Kikuchi, M., Dan, K. and Yoshio, K.: Seismic behavior of a reinforced concrete building due to large vertical ground motion in near source regions, *Proceedings of 12th. World Conference on Earthquake Engineering*, Paper No. 1867, 2000.
- 23) Wilson, J. C. and Liu, T.: Ambient vibration measurements on a cable-stayed bridge, *Earthquake Engineering and Structural Dynamics*, Vol. 20, pp. 723-747, 1991.
- 24) Usami, T. and Ge, H. B.: Cyclic behavior of thin-walled steel structures - numerical analysis, *Thin-Walled Structures*, Vol. 32, pp. 41-80, 1998.
- 25) Grondin, G. Y., Elwi, A. E. and Cheng, J.J.R.: Buckling of stiffened steel plates - a parametric study, *Journal of Steel Constructional Steel Research*, Vol. 50, pp. 151-175, 1999.

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斜張橋鋼製タワーの大地震時動的応答に関するパラメトリックスタディー

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斜張橋鋼製タワーの動的応答性状に与える減衰特性, 入力地震動, 初期不整, エネルギー吸収機構およびタワーの骨組形状などの諸因子の影響を調べるためにパラメトリックスタディーが実施される。大地震動による斜張橋鋼製タワーの非線形動的応答解析には有限要素法が用いられる。タワー形状により水平梁の位置や長さによっては動的応答に好ましくない影響を与えることが数値計算で示される。また, 建設時における初期不整の影響が明らかにされる。質量比例型減衰は鋼製タワー基部に発生する軸力および加速度応答を過大評価することが判明した。