Jurong Town Corp v Sembcorp Engineers and Constructors Pte Ltd
[2009] SGHC 93

Case Number : Suit 44/2006

Decision Date : 17 April 2009

Tribunal/Court : High Court

Coram : Chan Seng Onn J

Counsel Name(s) : Andre Maniam, Ong Pei Chin and Wong Baochen (Wong Partnership LLP) for the plaintiff; Soh Lip San, Elaine Tay and Tan Yu Inn Shannon (Rajah & Tann LLP) for the defendant

Parties : Jurong Town Corp — Sembcorp Engineers and Constructors Pte Ltd

Contract – Breach

17 April 2009

Chan Seng Onn J

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I. Background Facts

1 The plaintiff ("JTC") is the developer and owner of Woodlands Spectrum I ("Development"), which comprises some 17 blocks of 9-storey stack-up factories. The defendant ("SEC") was the main contractor for the Development under a contract dated 21 September 1998 ("Contract").

2 The Contract originally provided for the use of reinforced concrete ("r.c.") lintels/stiffeners for the brick walls in the Development. In the course of the Contract, SEC proposed to substitute steel lintels/stiffeners for r.c. lintels/stiffeners. JTC was amenable to that proposal. The term "stiffeners" is used in this judgment to refer generally to the lintels/stiffeners, which are not structural elements but fulfil the function of supporting and strengthening the walls. Stiffeners can be both horizontal and vertical. Lintels are horizontal beams built into the wall at the top of openings. Stiffeners can also be "lintels".

3 SEC engaged Dyntek Pte Ltd ("Dyntek") as its sub-contractor to design and construct the steel stiffeners. As between JTC and SEC, the design of the steel stiffeners was SEC's responsibility. The design was undertaken by Dyntek and its engineering consultants WP Brown. Mr Gary Wyatt ("Wyatt") of WP Brown, a Professional Engineer ("PE"), endorsed the design.

4 The Development was substantially completed in or around March 2000, but defects were observed thereafter at certain brick wall locations.

5 JTC said that these defects were caused by or attributable to the design and construction of the steel stiffeners. JTC therefore claimed damages, a declaration as to indemnification for future defects, interest and costs. The loss and damage suffered by JTC included the cost of rectification works.

6 By a direction made on 27 April 2007, the proceedings were bifurcated. Liability was to be determined first, and issues of quantum deferred to a later stage.

II. Legal / Factual Issues

A. SEC's Obligations

7 The obligations of a contractor who undertakes both the design and construction of particular works are well settled:

i. Hudson's Building and Engineering Contracts, at paragraph 4.066:

> A contractor undertaking to do work and supply materials impliedly undertakes…that both the workmanship and materials will be ***reasonably fit for the purpose*** for which they are required, unless the circumstances of the contract are such as to exclude any such obligation.

ii. Construction Law in Singapore and Malaysia, at page 72:

> A contractor undertaking a design responsibility warrants (as a question of fact rather than of law) that, where the purpose of the required construction has been adequately communicated to and accepted by him, the design will be ***fit for that purpose***.

iii. *MCST Plan No 1166 v Chubb Singapore Pte Ltd* [1999] 3 SLR 540 at paragraph 60:

> It is settled law that according to the principles of the common law, in a pure contract for work and materials the following warranties will be implied into it:
>
> (a) that the ***materials used will be of good quality*** (equivalent to merchantable quality in sale or supply of goods contracts);
>
> (b) that the materials are ***reasonably fit for the purpose*** for which they are used (equivalent to fitness of purpose of purchase in sale or supply of goods contracts).

iv. *Gema Metal Ceilings v Iwatani Techno Construction* [2000] SGHC 37 - the Court found at [69] that the plaintiffs were responsible to the defendants not only for the supply of materials but also design (of the metal ceiling system for the Kuala Lumpur International Airport) and held at [70] that:

The plaintiffs were aware of the requirements and specifications of the KLIA Project, and must be taken to have agreed to design a ceiling system which met those requirements.

8 Applying these well established principles to the present case, I found that SEC's obligations under the Contract were to ensure:

i. The suitability, adequacy, integrity, durability and practicality of the design of the steel stiffeners that SEC had proposed for substitution for the r.c. stiffeners;

ii. That the steel stiffeners it proposed were fit for the purposes for which they were intended;

iii. That the steel stiffeners were to be equal to or better than the r.c. stiffeners originally required under the Contract;

iv. That SEC would use all reasonable skill, care and diligence in the design of the steel stiffeners; and

v. That SEC would also construct the steel stiffeners using all reasonable skill, care and diligence and in a good workmanlike manner.

B. SEC's Breach and Resultant Damage

9 After careful consideration, I found on the evidence that SEC had breached its obligations in that:

i. the design of the steel stiffeners was lacking in suitability, adequacy, integrity, durability and practicality;

ii. the steel stiffeners were not fit for the purposes for which they were intended;

iii. the steel stiffeners were not equal to or better than the r.c. stiffeners originally required under the Contract;

iv. the steel stiffeners were not designed with reasonable skill, care and diligence; and

v. the steel stiffeners were not constructed with reasonable skill, care and diligence and in a good and workmanlike manner.

10 The steel stiffeners failed to support and strengthen the brick walls and were themselves deformed. In the circumstances, I found that JTC was entitled to rely on the principle of *res ipsa loquitur* - it was for SEC to justify the design and construction of the steel stiffeners, their fitness for purpose, and that they were at least equivalent to the r.c. stiffeners originally required. SEC had failed to do so.

C. SEC's Defence

11 SEC denied the obligations and breaches asserted by JTC. SEC sought to defend its design and construction of the steel stiffeners. It asserted that the defects in question were not caused by or attributable to the design or construction of the steel stiffeners, on the basis that:

i. SEC's proposal to use steel stiffeners only resulted in a substitution of the material used for the stiffeners, but the other aspects of the walls remained as designed by JTC;

ii. the steel stiffeners were approved and accepted by JTC and JTC's Superintending Officers ("SO");

iii. JTC, its officers and employees did, as SO, exercise independent judgment in making decisions relating to the design and specifications of the Development, including those associated with the steel stiffeners. With the inputs from JTC on the design of the steel stiffeners and with JTC's rigorous checking and approval system for the design, JTC had therefore placed little reliance, if any, on SEC with regard to the design of the steel stiffeners;

iv. in some areas where SEC alleged that r.c. stiffeners were installed, the walls had also bulged or cracked, evidencing that the problem lay with the design of the entire Development, rather than the design of the steel stiffeners;

v. SEC had executed and completed the Works "*to the satisfaction of the Superintending Officer*" as required under the Contract, and the SO had certified substantial completion of the Development on 8 March 2000 and issued the Final Completion Certificate on 31 October 2002; and

vi. the reports of Dr Chiew Sing Ping ("Dr Chiew") from Nanyang Technological University ("NTU"), and WP Brown evidenced that the steel stiffeners installed were sufficient to carry the design load of the Development.

vii. the fact that the defects surfaced some 4 years after construction indicated that the causes were time dependent. The loads due to the weight of the brick wall above were present even before the brick walls were plastered and hence were not time dependent, whereas the effects of creep and shrinkage of the r.c concrete structure were time dependent, and these, according to the SEC, imposed additional loads on the steel stiffeners and were therefore the cause of the defects. Apart from these effects due to creep and shrinkage, another shortcoming was the slender brick infill walls.

12 SEC asserted that the defects in Schedule 1 of the Statement of Claim "*could be caused by and/or attributable to and/or contributed by factors which were not within the purview or responsibility*" of SEC, for instance:

i. the Contract did not specify or require movement joints at the underside of the structure where the external brickwork infill walls and stiffeners abut;

ii. the thickness specified for the external wall in the Contract was inadequate for the height of the external wall; and

iii. fair wear and tear and/or lack of maintenance of the external walls in the Development.

13 SEC contended that the steel stiffeners were fit for their intended purpose, which was only to support the weight of the brick walls. The brick walls, not being structural components of the Development, were not expected to take additional loads from either the structure or the expansion of the brick walls. This assertion was set out repeatedly in SEC's closing submissions, for instance:

At [65]:

> As stated in Section (B)(i) above, the purpose of lintels, be it reinforced concrete lintels/stiffeners and/or Steel Lintels, is ***merely to carry the weight of the brickwall sitting immediately above a section of Steel Lintel***. As will be elaborated further in Section (D)(i)(a) below, ***since the calculations and laboratory test results show that the horizontal Steel Lintels, in the as-built condition, can carry as much as ten (10) times the weight of the brickwall above, the Steel Lintels have fulfilled their intended purpose***. (Emphasis added)

At [466]:

> It is submitted by the Defendant that this brings us back to the fact that, as acknowledged by the Plaintiff, the purpose of the horizontal Steel Lintels is <u>**ONLY**</u> to carry the weight of the brick wall sitting immediately above a section of the same. It has never been contended by the Plaintiff that the Steel Lintels are required to carry the extra forces induced by the effects of creep,

shrinkage and expansion. In fact, these effects have not even been considered by the Plaintiff's expert witness, Dr Ting, in his report where he makes no reference to such effects at all. (Emphasis added)

14 SEC submitted that its design duties in the present case ought to be limited to the design of the steel stiffeners which would fulfil their intended function if they did not deform and/or fail under the weight of the brick wall sitting immediately above the horizontal steel stiffeners. Since the brick walls were generally not intended to be load-bearing in a typical building, the additional loads imposed by the interaction of the parts of the Development designed by the plaintiff and the brick walls could not be expected to the accommodated by SEC's design of the of the steel stiffeners unless the presence of these loads had been specifically communicated to it, which was not the case. Since JTC had not provided SEC with the design calculations or the basis for design of the overall structure of the entire Development, there was no basis for JTC to make any assumptions concerning the additional loads which would be imposed on the brick walls due to the creep and shrinkage of the concrete as well as the expansion of the brick walls. SEC submitted that in the circumstances, it was reasonable for SEC to assume that the brick walls in the Development could be treated as non load-bearing walls.

D. Key Issues

15 The key issues in this case were largely factual and technical in nature:

i. whether the steel stiffeners were fit for their purpose;

ii. whether the steel stiffeners were equal to or better than the r.c. stiffeners originally required;

iii. whether there was negligence in the design and construction of the steel lintels/stiffeners; and

iv. whether the defects observed at areas with steel lintels/stiffeners were caused by and/or attributable to the design or construction of the steel lintels/stiffeners.

16 A subsidiary issue raised by SEC concerned SEC's own characterisation of JTC's pleaded case that the steel stiffeners were the sole cause of the defects observed at the Development and that that was the case JTC had to prove. According to counsel for SEC, JTC's case should be dismissed "*if any distress on site is attributable to the absence of the movement joints*", even if the steel stiffeners were defective. I agreed with JTC's submission that it could not be the law that a cause should be the "sole cause" in order for liability to be established. In any event, I did not think that SEC had correctly characterised JTC's case as had been pleaded by JTC.

17 On the whole, I agreed with the submissions of counsel for JTC. Accordingly, I adopted a large part of JTC counsel's well thought out and structured submissions, which included a careful analysis of the evidence (ably assisted by its expert) on many points in this judgment. I had earlier directed both counsel to ensure that the respective experts assist in the preparation of their closing submissions as this case involved very technical engineering considerations. I would like also to place on record my appreciation for the voluminous amount of extra work that both experts had to do in the

course of the trial in order to assist the court to better understand the various technical issues raised in respect of the cause of the defects found at the Development.

III. SEC'S Obligations

A. Fitness for Purpose

i. Whether SEC had an obligation to ensure that the steel stiffeners were fit for their intended purposes

18 SEC contended that an obligation to ensure that the steel stiffeners were fit for the purposes for which they were intended did not arise in this case as JTC did not rely upon SEC's skill and judgment for the same. SEC said that JTC was independently advised on the steel stiffeners, as evidenced by JTC's approval of SEC's submissions on the steel stiffeners.

19 In my judgment, any approval/acceptance of the steel stiffeners by JTC or the SO under the Contract would not relieve SEC of its own obligation to ensure that the steel stiffeners were fit for the purposes for which they were intended. In particular, clause 66 of the Standard Preliminaries for Construction Works provided that "*acceptance of any substitution shall in no way relieve the Contractor of any of his responsibilities for compliance with all the requirements of the Contract*". Further, clause 6.1 of the conditions of Contract stipulated (in relation to works designed by SEC) that: "*he [i.e. SEC] shall be fully responsible for the suitability, adequacy, integrity, durability and practicality of the design as set out in the Drawings [etc] submitted for the acceptance of the Superintending Officer…*". Clause 6.2 of the Conditions expressly stated that "*Acceptance by the Superintending Officer of such submission shall not relieve or in any way limit the responsibility of the Contractor under Clause 6.1*."

20 As regards the Superintending Officer:

i. Clause 2.1(2) of the conditions of Contract stipulated that "*Except as expressly stated in the Contract, the Superintending Officer shall have no authority to relieve the Contractor of any of his obligations under the Contract*";

ii. Clause 19 of the Standard Preliminaries stipulated that "*He [the SO Rep] shall have no authority to relieve the Contractor of any of his duties or obligations under the Contract*".

21 In view of the Contract terms highlighted above, I agreed entirely with the submission of counsel for JTC that SEC's reliance on the alleged independent judgment and decision making by JTC or its officers and employees to relieve itself of its own responsibilities under the Contract was entirely misplaced.

22 In any event, it was clear to me that the steel lintels/stiffeners were not designed by JTC, but by SEC and/or its subcontractor Dyntek. Further, there was clear reliance by JTC's "*officers and employees*" -- whom SEC identified as Mr Koh Cheng Hui, Mr Ng Eng Gim Gary ("Gary Ng") and Mr Tong King Yii ("Tong") -- on SEC to provide a stiffener system that was fit for its intended purpose.

23 In its particulars, SEC had specifically relied on 2 documents as evidence of the exercise of

independent judgment and decisions made by JTC, namely:

i. submission for approval made on 25 September 1998 dated 10 October 1998 (ref: JTC/STRUC/037A) in relation to *"stiffeners and tie beams"* from Dyntek - responded to by Tong on 10 October 1998; and

ii. submission for approval dated 16 April 1999 (ref: JTC/STRUCT/055) in relation to *"revised shopdrawing for Dyntek [stiffeners]"* - responded to by Gary Ng on the same day.

24 On the evidence, I found that neither Tong nor Gary Ng considered it their responsibility to review or approve alternative designs of the steel stiffeners. On the contrary, they relied on the PE designing the steel stiffeners (*i.e.* Wyatt) to ensure that the steel stiffeners were suitable and/or fit for their intended purposes.

25 Tong's response to SEC's submission for approval JTC/STRUC/037A in relation to "stiffeners and tie beams" from Dyntek was as follows:

> 1. The resistance performance shall comply fully with FSB's requirements for fire rated components.
>
> 2. Strengths and stiffness of steel vertical and horizontal stiffeners ***<u>shall be verified by PE</u>*** for all areas of usage.
>
> (Emphasis added)

26 Tong, himself an architect, was in any case in no position to evaluate the proposed steel stiffeners from an engineer's perspective. In any event, it was Tong's position that it was SEC's responsibility to ensure that the steel stiffener design was in fact verified by a PE (in this case, Wyatt of WP Brown, who endorsed the steel stiffener design). Indeed, SEC itself had acknowledged at [50] of its closing submissions that any review by Tong would be "*from an architectural point of view*".

27 With regards to the submission for approval dated 16 April 1999 (ref: JTC/STRUCT/055), Gary Ng stated that on or about 15 April 1999 he received a drawing entitled "*typical connection details (100mm walls)*", endorsed by Wyatt as PE. Parts of the drawing were clouded, with the comments *"IF GAP GREATER THAN 40mm PROVIDE FULL LENGTH SITE WELD TO TOP & BOTH SIDES"* and *"IF GAP GREATER THAN 40mm PROVIDE FULL LENGTH SITE WELD TO ALL FOUR FACES"*. These being the parts highlighted to Gary for his attention, they were the only aspects which he reviewed. His comments, "*Approved, pse give us endorsed calculations & drawings*" applied only in respect of the welding details proposed. Like Tong, Gary Ng relied on SEC's design, as endorsed by their sub-contractors' consultant engineer.

28 According to Gary Ng, the JTC/STRUCT/055 submission was submitted by SEC to regularise the paperwork in respect of his comment on this drawing.

29 The contemporaneous documents (namely, a fax from Dyntek to Mr Sim Beng Kiat ("Sim") (an ex-Project Manager of SEC) and Mr Chia dated 16 April 1999) also showed that Gary Ng's approval was understood by SEC and Dyntek to be in relation to intermittent welding, rather than the steel lintel/stiffener design as a whole.

30 From the totality of the evidence, I found that at all times it was SEC's obligation (not JTC's obligation) to ensure that the steel stiffeners were suitable and/or fit for their intended purposes regardless of any approval by JTC. SEC could not rely on its submissions of approval to JTC to relieve itself of its own obligation to ensure that the steel stiffeners were fit for their intended purpose. There was clear evidence (particularly from the testimony of Gary Ng and Tong as well as their responses to the submissions for approval cited by SEC in its closing submissions, which I do not find it necessary to set out in this judgment) that JTC was relying wholly on SEC, having undertaken both the design and construction of the steel stiffeners that it had proposed to JTC as substitution for the r.c. stiffeners, to provide a steel stiffener design that was suitable and/or fit for its intended purpose. Hence, JTC required that such design be endorsed by a PE. Accordingly, I rejected any suggestion from SEC that JTC had not placed reliance on SEC for the whole design and installation of the steel stiffeners proposed by SEC in substitution for the r.c. stiffeners.

31 I noted that Mr Ng Wee Beng, a director of Dyntek, had testified that he *"expected Mr Wyatt, as PE to come up with a design that is fit for the walls in question"*. Indeed, Ng Wee Beng made clear that Dyntek relied on WP Brown to design the stiffeners to be used for the Development as well as the end connections and that Dyntek would not check the calculations. In this regard, he said:

> Well, we are not engineers so we are not the wiser to really check it, which is why we engage them [i.e. WP Brown].

32 While Wyatt disagreed that the steel stiffeners were not fit for their purpose, he had not, at any point, contended that there was no obligation on him to design stiffeners that were suitable and/or fit for their intended purpose. Neither did he suggest that it was JTC's obligation to ensure the same.

33 JTC's spacing requirements of the stiffener system for the brick walls were for a vertical stiffener to be placed at every 5 m length and horizontal stiffeners to be placed at every 3.5 m height and Wyatt had understood these to be the "*maximum acceptable dimensions*". Wyatt did not execute a standard design for the steel stiffeners based on these wall dimensions. Rather, his "*design was always done for the specific loading conditions that were applicable*". Indeed, he exercised his own judgment in determining whether horizontal steel stiffeners were required even if JTC spacing requirements were exceeded. Without seeking JTC's concurrence to exceed the nominal 3.5m x 5 m spacing, Wyatt in fact took the position that no stiffener was necessary for certain walls of 3.6m, 3.62m and 3.9m heights at the SS 1000 series of the Development. In the circumstances, I found that Wyatt had undertaken design responsibility for the steel stiffeners.

34 Accordingly, I accepted the submission of JTC's counsel that SEC was, in fact and in law, obliged to ensure that the steel stiffeners were suitable and/or fit for their intended purpose. Further, on the evidence I found that there was clear reliance by JTC (and its officers and employees) on SEC (and its PE) to provide the alternative steel stiffener system that was fit for its intended purpose. The design responsibility was in fact undertaken by Wyatt for SEC.

ii. The purpose for which the steel stiffeners were intended

35 SEC's witnesses, Wyatt and Mr Phillip J. Jones ("Jones"), the expert witness for SEC, acknowledged that a stiffener system should prevent lateral buckling of a brick wall *i.e.* ensure wall stability. This was necessarily premised on both the vertical and horizontal stiffeners having greater lateral stiffness than the brick wall. The design of the stiffener system thus had to cater for all lateral forces including those from wind loads. Wyatt himself admitted in his evidence that he did design the stiffeners to withstand wind loads. I thus rejected the submission from SEC that JTC did not require

the stiffeners to carry wind loads. That submission was without evidential basis.

36 In addition to providing lateral restraint to the brick wall, the stiffeners would also need to be able to withstand the weight of the brick wall above the horizontal stiffeners (taking into account the various loads acting upon the stiffeners). At a minimum, the stiffeners should not unacceptably deflect or deform (let alone fail) after being sandwiched between the lower brick wall and the brick wall above and incorporated as part of the wall. JTC required the stiffeners to bear the vertical load even if the brick walls below were removed. Hence, the steel stiffeners were to be "load-bearing". JTC thus required the horizontal stiffeners to be able to function as beams should the brick walls below them be removed and also as typical brick wall stiffeners sandwiched between the upper and lower brick walls.

37 Sim (SEC's then project manager), Ng Wee Beng (Dyntek) and Wyatt all acknowledged this in their evidence.

38 In essence, therefore, besides providing lateral stability to the brick walls, the horizontal stiffeners should be able to function as beams, transferring the weight of the brick wall above it to the support at the two ends, which could be columns (if they were sufficiently near) or vertical stiffeners.

39 Self-evidently, the steel stiffeners could not be considered to work if they could only carry the "*design load of the brickwalls (sic) of the Development*"- a pure theoretical construct - but not the actual loads.

40 Accordingly, I rejected SEC's contention that the purpose of the horizontal steel stiffeners was **<u>only</u>** to carry the weight of the brick wall above it. SEC itself admitted at paragraph 23(b) of its closing submissions that "*the purposes of stiffeners and / or lintels is to <u>stiffen the brickwalls within which they are installed</u>*". To serve the purpose of stiffening the brick wall within which the stiffeners were installed, the stiffeners had to adequately stiffen the actual walls, bearing both vertical and lateral loads, besides being able to carry the weight of the brick wall above it. I noted the evidence of the following witnesses, which showed that the purpose of the steel stiffener was not simply and solely to carry the weight of the brick wall immediately above it:

i. The evidence of Dr Ting Seng Kiong ("Dr Ting"), the expert for JTC, that stiffeners "*fulfil the function of supporting walls (typically brick walls)*";

ii. Gary Ng's evidence that stiffeners were "*required to make the wall 'stiffer'*"; and

iii. Tong's evidence that the purpose of a stiffener / lintel was "*[t]o stiffen the brick wall to make sure that it's stable*".

41 I accepted JTC's submission that SEC was effectively seeking to sustain the unreal position that the steel stiffeners were fit for their purpose as long as they worked **<u>in theory</u>** to support a free standing wall isolated from the building within which they were found (as if the weight of the brick wall was the only possible load on the steel stiffeners), even if **<u>in fact</u>** such steel stiffeners when installed on site, would unacceptably deform and cause damage to the wall finishes when subjected to the **<u>actual lateral and vertical loads</u>**.

a. Serviceability

42 First, it was clearly insufficient that the stiffeners merely had sufficient strength to carry the load (design load or otherwise). For a stiffener to work, it should be able to carry the load without unacceptable deformation or deflection. Otherwise, there would be damage to the wall.

43 Dr Chiew, the expert witness called by the JTC, put it pithily:

> *When we do design, we don't just think in terms of the strength. Strength is important to us; you must be able to transfer and carry the load. But equally important in service, the deformation, the deflection must not be excessive, such that it is noticeable or it causes other defects*.

44 Any suggestion that the stiffeners remained fit for their purpose as long as they were strong enough to carry the load of the brick walls, even if the stiffeners would themselves deform (by buckling, twisting or side-sway) and the wall finishes would be damaged would certainly be contrary to all common sense and reason.

45 Indeed, when considering the deflection limits that were acceptable to him, Jones, the expert called by SEC himself highlighted that the "*deflection should not be so great as to cause damage to the applied finishes*".

b. Expansion of bricks

46 Second, the stiffeners were intended to be installed in brick walls. The designer for the brick wall stiffeners ought to know that bricks would expand over time and hence, ought to have factored that into the design. As such, in designing the stiffeners, any additional loads / stresses that might be imposed on the stiffeners as a result of such brick expansion had to be accounted for.

47 Wyatt acknowledged both the fact that "bricks grow" and that additional stresses arising from such brick expansion would result. But he simply dismissed these effects on the basis that "*if you isolate the brickwork from the structure, then you also achieve relieving these stresses due to brick growth*".

48 If this was in fact a cause of the defects, I could not see how that would absolve SEC of liability, when its design of the steel stiffeners failed to account for the additional compressive stresses on the brick work resulting from this known phenomenon of brick expansion.

49 Indeed, on Jones' own account, expansion of the brick work accounted for more than 70% of the total differential movement of 2.7 mm of the r.c. structure and the brick work in a 3.05 m high brick wall. At page 237 of D-29, the total movement was broken down as follows:

i. shrinkage and creep of concrete (after 6 years) = 0.79 mm

ii. moisture movement of brick work = 1.53 mm; and

iii. thermal movement of brick work = 0.38 mm.

50 I agreed with the submission of counsel for JTC that it was no excuse to say that SEC could

not have accounted for such brick expansion as it had no knowledge as to the assumptions made in the design of the overall structure. Even if (for the sake of analysis) the structure were ignored, Wyatt was still fully aware of the dimensions of the brick panels. More fundamentally, SEC could well have chosen not to substitute the r.c. stiffeners with steel stiffeners if SEC was not in a position to provide a steel stiffener system that worked.

51 Indeed, it was noteworthy that Dyntek itself understood that the onus was on Wyatt to raise questions if more information was required in the design of the steel stiffeners. Ng Wee Beng had testified in this regard that if additional information was required to provide a good design, it was for WP Brown to ask all the necessary questions.

COURT:	I'm saying about sufficiency of design. I'm not saying about agreement or anything; I'm saying about sufficiency of design, that he fails to ask any question that would be relevant to him providing a good design, and he puts his chop there, as far as you and WP Brown is concerned --
A.	Of course. That is his responsibility, to ask --
COURT:	Ask all the necessary questions?
A.	I believe that should be the way.

c. Brick walls did not have movement joints

52 As a corollary, it was similarly no excuse for SEC to say that the installation of movement joints would have accommodated the compressive stresses from the movement in the r.c. structure and the brickwork and prevented the defects that were seen on the Development.

53 It was not JTC's case that the lack of movement joints was a cause of the defects on site. If however, it were to be found to be a cause, then JTC's position was that SEC should not be absolved of liability for the failure of its steel stiffeners to accommodate such additional stresses resulting due to the lack of movement joints.

54 Wyatt was well aware that the brick walls in the Development would not have movement joints when he undertook the design of the steel stiffener system. It was for him to design a steel stiffener system that worked for such walls, and not to design steel stiffeners that required the provision of movement joints to work when he knew full well that these were not installed on site! I agreed with the submission of counsel for JTC that SEC was effectively asking to be absolved of liability for designing stiffeners that were doomed to fail. SEC's position was that the steel stiffeners were not designed to work without movement joints even though it knew, before design of the steel stiffener system commenced, that no movement joints were to be provided by JTC. This position was, quite clearly, untenable.

55 In fact, Wyatt conceded that if he had foreseen the compressive loads induced in the brick work, he would have provided for horizontal compressive joints.

56 I noted that under clause 66 of the Standard Preliminaries, the Contractor (*i.e.* SEC) "*shall also be fully responsible at his own cost and expense for any additions to other parts of the Works resulting from the substitution and for any delay arising*" from a substitution of materials. Applying clause 66 of the Standard Preliminaries to the present facts, if it was necessary for movement joints

to be installed in the brick walls for the steel stiffeners to work, it clearly was SEC's obligation to provide the same, not JTC's. It should be highlighted in this regard that SEC's own witness, Sim, confirmed that in his experience, horizontal compressive joints not provided in brick walls with r.c. stiffeners did not cause any problems.

57 This was consistent with Tong's evidence that it was "*usual practice*" that a contractor would fill the gap between the brick wall and the underside of the beam with mortar, and this served as the expansion joint.

58 Dr Ting opined the same in paragraph 8.1(i) of his report, where he stated, *inter alia*, that:

> The provision of movement joints at the bottom of lintels is not required. It is standard construction practice in Singapore for a contractor to in-fill the stiffener/lintel-brick wall junction/interface with a weak mortar. This serves to accommodate movement of the stiffeners or beams.

59 I thus rejected SEC's defence, which sought to attribute the problems with its steel stiffeners on the absence of movement joints.

60 In summary, the steel stiffeners in the present case would be fit for their intended purpose only if they:

i. stiffened / stabilised the brick walls by carrying wind load;

ii. could carry the actual weight of the brick wall above them when the bricks beneath them were to be removed;

iii. did not themselves deform or cause damage to the wall finishes when carrying the lateral and vertical loads above; and

iv. worked even if no movement joints were provided in the brick walls.

61 I rejected the assertion of SEC that the fitness of the steel stiffeners for their intended purpose was to be evaluated with respect to only one aspect *i.e.* their ability to support the weight of the brick walls, without any consideration of the steel stiffener system being installed as a part of the building to stiffen the walls against all reasonably expected loads *and* without consideration of the need to comply with the relevant building codes. In my view, the stiffener system would not be fit for its purpose if it only satisfied one design aspect but not other relevant design aspects. All design aspects relevant to a stiffener system should be satisfied before it could be said that the stiffener system was fit for its intended purpose. It was clearly insufficient merely to show that the steel stiffeners could *per se* carry the weight of the brick wall above, if the steel stiffener system would not work if other normal anticipated conditions, factors or loads were taken into account. It must be borne in mind that SEC's obligation was to design and construct a working practical stiffener system that met the relevant building code requirements and satisfied all other contractual requirements as might have been stipulated by JTC. In my view, it would be an extraordinary contention that a designer of the steel stiffeners could turn a blind eye to all the expected loading conditions of the steel stiffeners as installed in the walls. Obviously the designer of the steel stiffener should take into account "*additional forces*" (*i.e.* beyond the weight of the brick wall) that were "*in the realm of normal civil engineering knowledge*". If the stiffener system was not able to function satisfactorily as

a stiffener without the movement joints, the designer could either design a stronger stiffener system to work without movement joints, or himself provide the movement joints if the stiffener system as designed could not work satisfactorily without the movement joints. In either event, the responsibility of providing the movement joints lay with SEC and not JTC on the facts of this case. It was no excuse for SEC to say that if only JTC had provided for movement joints, the steel stiffeners (which as designed by Wyatt needed movement joints before they could perform satisfactorily as wall stiffeners) would have been fine. JTC was obviously not obliged to make changes to the Development to accommodate inadequacies in SEC's steel stiffeners; rather it was for SEC to take the Development as a "given", and design and construct steel stiffeners that would work and not themselves deform or cause damage to the wall finishes when carrying the lateral and vertical loads. Indeed, it was notable that Wyatt himself had conceded that if he had foreseen the magnitude of the compressive stresses in the brick walls, he would have provided for movement joints.

B. Equivalence / Design / Construction

i. JTC's Position

62 Clause 4.1 of the Standard Conditions of Contract expressly provided that:

> The Contractor [*i.e*. SEC] shall, with due care and diligence, design (to the extent provided for by the Contract), execute and complete the Works and remedy any defects in the Works in accordance with the provisions of the Contract and to the satisfaction of the Superintending Officer. The Contractor shall provide all superintendence, labour, Plant, Construction Equipment, materials, goods and all other things, whether of a temporary or permanent nature required in and for such design, execution, completion of the Works and remedying of any Defect. Nothing in this Clause shall affect the Contractor's responsibilities under common law to complete the Works.

63 Further, clause 6.1 of the Standard Conditions of Contract provided that:

> Where the Contract expressly provides that the whole or any part of the Permanent Works shall be designed by the Contractor, he shall be fully responsible for the suitability, adequacy, integrity, durability and practicality of the design as set out in the Drawings, Specifications, manuals, calculations and other information submitted for the acceptance of the Superintending Officer under Clauses 6.2 and 6.3, including any subsequent amendment of such design.

64 Substitution of materials was specifically addressed in clause 66 of the Standard Preliminaries for Construction Works, pursuant to which the proposed substitution was to be:

> …equal to or better than that specified or accepted by the Corporation at the time of the award… Acceptance of any substitution shall in no way relieve the Contractor of any of his responsibilities for compliance with all the requirements of the Contract. He shall also be fully responsible at his own cost and expense for any additions to other parts of the Works resulting from the substitution and for any delay arising therefrom.

65 Compliance with relevant standards was also expressly provided for in note 2 of the Standard Specification for Construction Works, which stated as follows:

> Unless otherwise specified, all works and materials shall comply with the relevant Singapore Standards as laid down by the Singapore Institute of Standards and Industrial Research. Where this is not possible, the relevant British Standards or other approved standards shall apply. All standards referred to in the Standard Specification shall be those editions (including

amendments) current thirty days before the closing date of tender.

66 Since SEC proposed to substitute the original r.c. stiffeners with steel stiffeners to be designed and constructed by their sub-contractor Dyntek, it followed that as between SEC and JTC:

i. SEC was fully responsible for the suitability, adequacy, integrity, durability and practicality of the design of the steel stiffeners;

ii. the steel stiffeners were to be constructed with all reasonable skill, care and diligence and in a good and workmanlike manner; and

iii. the steel stiffeners were to be equal to or better than the r.c. stiffeners.

67 Further or alternatively, SEC owed JTC a duty of care in tort:

i. to use all reasonable skill, care and diligence in the design of the steel stiffeners; and

ii. to construct the steel stiffeners using all reasonable skill, care and diligence and in a good and workmanlike manner.

68 It was also SEC's obligation to ensure that the stiffeners complied with the relevant standards applicable.

ii. SEC's Defence

69 SEC denied the obligations asserted by JTC. SEC submitted that it had complied with clause 4.1 of the conditions of Contract, as evidenced by:

i. the SO's issuance of the Certificate of Substantial Completion and the Final Certificate of Completion; and

ii. the SO's letter dated 24 February 2006.

70 In my view, the Contract terms made it clear that any such approval / acceptance did not relieve SEC of its obligations. Further, from clauses 17 and 18 of the conditions of Contract, "*substantial completion*" did not mean that SEC had completed the entire works or that what SEC had done was free of defects. Clause 18.5 specifically provided that the stipulations in clauses 18.1 to 18.4 as to defects liability "*shall not derogate in any way whatsoever from the Contractor's liability under the Contract or otherwise for defective work at common law*."

71 Neither would the issue of the Final Completion Certificate preclude JTC from bringing this claim. Clause 33.2 specifically provided that:

> No certificate of the Superintending Officer shall of itself be conclusive evidence that the Works have been completed or that any Plant, materials, goods or work to which it relates are in accordance with the Contract.

72 Insofar as SEC had sought to rely on the letter from Jurong Consultants Pte Ltd ("JCPL") dated 24 February 2006, that letter merely expressed the view that the SO was *functus officio* after the Final Completion Certificate, and that the SO "*would not therefore be in any position to act/decide on any matters under the Contract*". JCPL and the SO did not express an opinion on the merits of JTC's claim.

73 SEC also contended that it had complied with clause 66 of the Standard Preliminaries as:

i.	the proposal for the substitution of the r.c. stiffeners with steel stiffeners was reviewed by, commented upon and approved by JTC, JTC's Technical Services Group, the SO and/or the SO's Representatives;
ii.	SEC complied with the requirements of the Contract and duly considered and abided by: (a) the detailed specification prepared by JTC for the r.c. stiffeners; (b) the comments on and requirements dictated by JTC's servants and/or agents and the SO in respect of SEC's submissions on the steel stiffeners; (c) JTC's servants and/or agents and the SO's approval of SEC's submissions on the steel stiffeners.

74 I rejected the above contention of SEC. Clause 66 of the Standard Preliminaries expressly stated that "*acceptance of any substitution shall in no way relieve the Contractor of any of his responsibilities for compliance with all the requirements of the Contract*." SEC's reliance on "*approval*" by JTC in relation to SEC's proposed substitution of r.c. stiffeners with steel stiffeners was therefore misconceived.

75 Indeed, Sim agreed that "*since SEC was substituting steel lintels or stiffeners for the RC lintels or stiffeners originally required, the steel ones would have to be at least equal to or better than the RC ones*".

76 SEC also submitted that Clause 6.1 of the Standard Conditions of Contract was not applicable to the steel stiffeners as the Contract did not expressly provide that the steel stiffeners were to be designed by SEC.

77 It was not disputed that the Contract did not expressly provide that SEC was to design the steel stiffeners. However, the standard required of any work "*designed by the Contractor*" under Clause 6.1 must, by necessary implication, apply to work for which the contractor (*i.e.* SEC) had proposed an alternative design which was accepted by JTC.

78 I accepted JTC's submission that it was an implied term of the Contract that where SEC had

proposed a design in substitution for a design under the Contract, SEC would be fully responsible for the suitability, adequacy, integrity, durability and practicality of the substituted design that it had proposed.

79 Having set out the scope of SEC's obligations, I now set out the facts relating to SEC's breaches and explain why I found SEC to be liable for the defects as claimed by JTC.

IV. SEC'S Breaches

A. Design

80 It was not disputed that the general standard applicable to the steel stiffeners was British Standard 5950 Part 5: Code of practice for design of cold formed thin gauge sections ("BS 5950-5").

81 JTC's position was that the design of the steel stiffeners was in breach of various requirements under BS 5950-5, in that:

i. the lateral deflections of the vertical stiffeners exceeded the limit prescribed by section 5.7 read with section 2.4.2;

ii. the design for the H1-250 horizontal steel stiffeners did not satisfy the lateral buckling requirement of section 5.6.1;

iii. there was inadequate consideration of the effects of torsion, as required by section 5.9 of BS 5950-5;

iv. the maximum slenderness for the horizontal stiffeners, H1-250 and H1-300 exceeded the limit specified in section 6.2.2 of BS 5950-5.

82 At this juncture, it might be worthwhile for me to state that non-compliance of the design of the steel stiffeners with the applicable provisions of BS 5950-5 would in and of itself constitute a breach of note 2 of the Standard Specification for Construction Works. In addition, such non-compliance would also be a breach of SEC's obligation to use all reasonable skill, care and diligence in the design of the steel stiffeners, for which SEC would be liable to JTC.

i. Sections 5.7 and 2.4.2 of BS5950-5 : Deflection

83 Section 5 of BS 5950-5 related to "*Design of members subject to bending*".

84 SEC's expert witness, Jones, agreed that this section applied to both the horizontal and vertical steel stiffeners. Gary Wyatt, when he was confronted with the relevant sections on deflection limits in the code on steel, section 5.7 read with section 2.4 of BS 5950-5, also agreed that BS 5950-5 dealt with both vertical and horizontal deflections, and that the deflection limits stated in the code took into account all loads regardless of the direction they came from.

85 Section 5.7 specifically addressed the issue of deflection, which would be related to the stiffness of an element *i.e.* how much an element would deflect or deform under load. A stiffer element would experience lesser deflection under the same load.

86 Section 5.7 of BS 5950-5 provided that "*[t]he recommended deflection limitations for beams are given in [section] 2.4.2*".

87 Section 2.4.2, in turn, stated that:

> Deflection
>
> The deflection under serviceability loads of a building or its members should not impair the strength or efficiency of the structure or its components or cause damage to the finishings.
>
> When checking the deflections the most adverse realistic combination and arrangement of unfactored loads should be assumed, and the structure may be assumed to be elastic.
>
> Table 3 gives recommended deflection limits for certain structural members. Circumstances may arise where greater or lesser values would be more appropriate. …

88 Table 3 provided for a deflection limit of span/360 for "*beams carrying plaster or other brittle finish*" for *"deflection of beams due to unfactored imposed loads*".

a. Deflection limit to be complied with - JTC's position

89 JTC's position was that this deflection limit of span/360 was applicable to the stiffeners, both in respect of vertical deflection and horizontal deflection, and regardless of the type of loads imposed (be it dead load, imposed load or wind load). According to JTC, the steel stiffeners had failed to comply with this deflection limit of span/360 provided in BS 5950-5.

b. Deflection limit to be complied with - SEC's Position

90 SEC's expert witness, Jones, did not accept that the deflection limit of span/360 set out in BS 5950-5 was applicable to the steel stiffeners installed at the Development.

1 VERTICAL DEFLECTION

91 In respect of vertical deflections, Jones' initial opinion was that "*both concrete and steel horizontal stiffeners comply with code requirements for deflection*". At the start, Jones confirmed on the stand that by "*code requirements*" he was referring to BS 5950-5. When clarification was sought on the deflection limits stipulated by BS 5950-5 in relation to the steel stiffeners in the Development, Jones changed his position by declaring that there was a "*well-known problem*" in BS 5950-5 that it did not provide deflection limits for vertical deflection due to dead load, and that his statement on page 19 of D-29 should really read "*both concrete and steel horizontal stiffeners comply with my own requirement which is based on reasonable engineering judgment*".

92 Jones contended that the deflection limit of span/360 in BS 5950-5 applied only to an imposed load. As there was no imposed load, Jones considered that BS 5950-5 allowed unlimited deflection.

93 In Jones' view as such, BS 5950-5 did not set down any specific deflection limit arising from dead load or transient load *e.g.* wind load on the steel stiffeners. There was only "*an implied code requirement that the deflection should not be so great as to cause damage to the applied finishes*".

94 His personal view was that a deflection limit of span/150, or a range between 1/150 and 1/180 would be acceptable. He considered this to be "*the limit at which you would then start to get cracking of render and plaster finishes, dependent upon the flexibility of the applied finish itself, whether it's got additives in to improve its flexibility*".

95 However, Wyatt, the designing PE, applied a more exacting vertical deflection limit of span/325 in his design.

96 This ratio was derived from section 7.1.2(b) of British Standard 5977 Part 2: Specification for prefabricated lintels ("BS 5977-2") (P-31), which he considered to be more appropriate than BS 5950-5 as it was specifically related to lintels.

97 Section 7.1.2(b) of BS 5977-2 provided that:

> The manufacturer shall prove by design from first principles (see 7.2) or by design based on test (see 7.3) that the lintel… (b) will deflect in either plane not more than 1/325 times the effective span when subjected to the stated safe working load.

2 HORIZONTAL DEFLECTION

98 In respect of horizontal loading, however, Wyatt applied a deflection limit of span/50 in his design.

99 He contended that the ratio of span/325 applied in respect of lateral deflection due to vertical loading *only* since "*safe working load*" in section 7.1.2(b) of BS 5977-2 referred only to the vertical load. He insisted that there was no deflection limit under BS 5977-2 for lateral deflection from horizontal loading as the ratio of span/325 did not apply when the lateral deflection was due to a horizontal load. Effectively, Wyatt contended that a steel stiffener design could be non-compliant with section 7.1.2(b) of BS 5977-2 if the lateral deflection arising from vertical loads exceeded span/325, but the same level of lateral deflection would have been acceptable if the deflection arose from horizontal loads. I did not find this contention amenable to reason.

100 Firstly, it should be noted that BS 5977-2 referred specifically to lintels over openings, and not brick wall stiffeners more generally. This was not disputed by Wyatt.

101 When confronted with the relevant sections on deflection limits in the code on steel, section 5.7 read with section 2.4 of BS 5950-5, Wyatt agreed that BS 5950-5 dealt with both vertical and horizontal deflections and the deflection limits stated in the code took into account all loads regardless of the direction they come from.

102 In any event, even SEC's expert, Jones, would not endorse a deflection limit of span/50 and applied a range of span/150 to span/180.

103 While Jones maintained that there was no direct code requirement of limit for deflection of stiffeners horizontally under wind load in BS 5950-5 (which JTC disagreed with), he adopted the *same range* of span/150 to span/180 in *both* vertical and horizontal deflections (whether from vertical or horizontal loading). Implicit in this was that the adoption of different limits for deflections in the vertical and horizontal directions or for vertical and horizontal loads would not comply with reasonable engineering judgment.

c. Compliance of the steel stiffeners with deflection limits

104 Jones calculated the lateral deflections of certain vertical steel stiffeners using various wall dimensions, and applied a wind load of $0.5kN/m^2$ for external walls and $0.2kN/m^2$ for internal walls (which wind loads were agreed between the experts as well as Wyatt).

105 These were reflected in D-42 and the results indicated that even applying the ratio of span/150 to span/180, the lateral deflections of the vertical steel stiffeners exceeded the allowable ratio.

106 Of the 5 situations considered, 4 were of actual conditions on site:

i. an external wall in the SS 2000 series;

ii. an elevation 1 external wall in unit #03-12;

iii. an internal wall in the same unit, #03-12; and

iv. an external wall of dimensions as set out in page 335 of Jones' 1st AEIC .

107 The other calculation was for theoretical wall dimensions of 7 m length by 10 m height, with a cross in the middle where the vertical and horizontal stiffeners intersected, as derived from JTC's requirement that the brick walls be strengthened with "*r.c. stiffeners at every 5 metres length & r.c. beams at every 3.5 m height*".

1. EXTERNAL WALL IN THE SS 2000 SERIES

108 The vertical stiffener used at this external wall panel was a V1-200. The height of the vertical stiffener was 5250 mm. The horizontal stiffeners on either side of the V1-200 vertical stiffener were H1-300 and were located at a height of 2200 mm from the floor and 3050 mm from the soffit of the beam. Jones adopted a left span of 3600 mm and right span of 4500 mm in his calculations.

109 Jones' calculations reflected a 38.9 mm lateral deflection of the stiffener in that wall dimension. This produced a deflection ratio of height/132, which failed even the reduced requirements of Jones, of a ratio of height/150 to height/180. Jones himself agreed as much.

2. #03-12 ELEVATION 1 EXTERNAL WALL

110 The vertical stiffener used at this external wall panel was a V1-100, with a height of 5600 mm. The horizontal stiffeners on either side were H1-250, each of 4000 mm span and located at a height of 2200 mm from the floor and 3100 mm from the soffit of the beam.

111 Jones' calculations indicated a 54.4 mm lateral deflection of the stiffener in that wall dimension. This was equivalent to a deflection ratio of height/97, which was outside what even Jones would consider acceptable.

3. #03-12 INTERNAL WALL

112 The vertical stiffener used at this internal wall panel was a V1-50. The horizontal stiffeners on either side were H1-150, each of 3100 mm span and located at a height of 3100 mm from the floor and 2500 mm from the soffit of the beam.

113 There was some disagreement between the parties on the height of the vertical steel stiffener, which had been inspected by JTC's expert as well as Mr John Ogilvie McGowan ("McGowan"), a

representative of Babtie Asia Pte Ltd ("Babtie Asia").

114 Dr Ting used a height of 5600 mm (rounded up from 5580 mm, the height set out in relevant drawing); Jones insisted that the height was 4450 mm and calculated the deflections on that basis. SEC based this on its assertion that there was a beam at the top of the brick wall and the vertical steel stiffener had terminated at the beam - an assertion that SEC was unable to substantiate with any drawing but only a photograph, which did not even show that the vertical stiffener had terminated where SEC asserted was a beam.

115 Based on this alleged reduced height of 4450 mm, Jones calculated a 19.2 mm deflection of the V1-50 stiffener. This was equivalent to a deflection ratio of height/231 and which would comply with Jones' own deflection limits of height/150 to height/180 (although it would still not meet the deflection ratio of span/360 provided in BS 5950-5). If the wall height of 5600 mm (as reflected in the drawing at AB 9881) was used in the calculations, the lateral deflection would be 47.1 mm, which produced a deflection ratio of height/119, falling outside Jones' acceptable deflection limits.

4. PAGE 335 OF JONES' 1ST AEIC

116 The vertical stiffener used at this external wall panel was a V1-200, with a height of 5250 mm. The horizontal stiffeners on either side of the V1-200 vertical stiffener were H1-300 each of 4400 mm span and were located at a height of 2200 mm from the floor and 3050 mm from the soffit of the beam.

117 Jones' calculations reflected a lateral deflection of the stiffener of 43.9 mm (using the second moment of area of the cross section "I" = 2.35×10^6) and 44.3 mm (using I = 2.33×10^6). This was equivalent to a deflection ratio of height/119 and height over/118 respectively. These did not comply with even the reduced requirements countenanced by Jones, of a deflection ratio of height/150 to height/180. Jones conceded that.

5. 7M BY 10M THEORETICAL WALL PANEL

118 The vertical stiffener used in this theoretical wall panel was a V1-200, with a height of 7000 mm. The horizontal stiffeners on either side of the V1-200 vertical stiffener were H1-300 each of 5000 mm span and were located at a height of 3500 mm from the floor and from the soffit of the beam.

119 Jones' calculations reflected a lateral deflection of the stiffener of 155.6 mm (using pinned connections, which was the assumption used in the calculations of all the above 4 scenarios) and 113.5 mm (if partial moment restraint was assumed). This produced a deflection ratio of height/45 and height/62 respectively, which fell far short of the reduced requirements of Jones, of a deflection ratio of height/50 to height/180. Jones himself agreed that the deflection ratios in this scenario were "*well outside [his] limit*".

120 It was common ground between both parties as such that the lateral deflections of the vertical stiffeners were unacceptable. Jones conceded that at the deflection limit of span/150 to span/180, "*you would start to get cracking of render and plaster finishes*", which was one of the mode of defects observed on site.

121 In my judgment, the steel stiffeners failed to comply with the deflection limit of span/height over 360 stipulated in section 5.7 read with section 2.4 of BS 5950-5, and this constituted a breach of SEC's obligations to JTC.

ii. Section 5.6 of BS 5950-5: Lateral Buckling

122 Section 5.6 of BS 5950-5 related to lateral buckling. Section 5.6.1 provided, *inter alia*, that:

> Lateral buckling, also known as lateral-torsional buckling, will not occur if the beam is adequately restrained against lateral movement and twisting. Restraints may be deemed to provide adequate strength if they are capable of resisting a lateral force of 3% of the maximum force in the compression flange or chord, divided equally between the points of restraint, subject to a minimum force of 1% per restraint.

123 JTC's position was that the design for the H1-250 horizontal steel stiffeners did not satisfy the lateral buckling requirement of section 5.6.1 of BS 5950-5.

124 Jones accepted that a section 5.6 check was necessary if one were designing steel stiffeners for a brick wall up to 3.5 m height and 5 m length and he would have done such checks himself. Wyatt, however, did not check the steel stiffeners against section 5.6 of BS 5950-5 when he was executing the design.

125 In paragraph 7.1 of exhibit "PJJ-2" to his 1st AEIC, Jones stated that:

> By considering the lateral buckling behaviour of the steel stiffeners as stated in Clause 5.6.1 of BS 5950-5:1998, if no restraint is provided, inadequate buckling resistance movement capacity arises for stiffener H1-250.

126 Jones confirmed that by this statement, he meant that the H1-250 steel stiffener, as designed, failed to comply with Section 5.6.1 of BS 5950-5.

127 As such, I concluded that the steel stiffeners had failed to comply with the lateral buckling requirement under BS 5950-5, and accordingly this constituted another breach of SEC's obligations.

128 While Jones sought to rely on the brick walls and the vertical stiffeners to provide lateral restraint, he did not perform any calculations to establish quantitatively that this would be sufficient to counter the weakness of the steel stiffeners in terms of lateral buckling.

iii. Section 5.9 of BS 5950-5: Effects of Torsion

129 Section 5.9 related to the effects of torsion. Section 5.9.1 stated that:

> General
>
> Where possible for open sections, the effects of torsion should be avoided either by the provision of restraints designed to resist twisting or by ensuring that lateral loads are applied through the shear centre.

130 The horizontal steel stiffeners were designed with approximately 75 mm of the bottom flange cut out at both ends *i.e.* it was not a box-section at the end of the stiffeners, but an inverted U-section. Jones agreed that for the purposes of analysis, the end sections could be considered open sections.

131 It followed that consideration should be given to section 5.9.1 of BS 5950-5 in designing the steel stiffeners.

132 In my judgment, the steel stiffeners designed with these open sections at the ends were weaker than closed sections for both torsional loads and lateral loads. As the torsional capacities of the end sections were low, the likelihood of torsional buckling due to eccentricity of vertical loads was present for H1-250 and H1-300. The design of the steel stiffeners overlooked their rotational deformations such that any slight eccentricity would cause the steel stiffeners to twist and warp. Accordingly, I agreed with JTC's contention that SEC had failed to comply with section 5.9 of BS 5950-5. In my view, any effects of deformation and twisting due to and at the end conditions would be geometrically magnified when the actual lateral deflection or rotational movements at the centre of the steel stiffeners were to be considered. Hence, SEC's contention that the open sections were only at the end and not throughout the entire length of the stiffener (and therefore would have minimal or no effect) ignored the geometrical effect of magnification that the end deformations would have at the centre of a long steel stiffener.

133 Dr Ting calculated a torsional moment of 0.068 kNm and a torsional capacity of 0.0632 kNm in respect of the H1-250 steel stiffener. His calculations showed that the torsional moment exceeded the torsional capacity of the section. At the support location of the steel stiffeners, only a maximum eccentricity of 4.7 mm in the loading could be allowed before the torsional capacity of the section would be exceeded. These calculations confirmed that the designed torsional capacities of the section were inadequate.

134 In respect of the H1-300 steel stiffener, Dr Ting calculated a torsional moment of 0.097 kNm and a torsional capacity of 0.1176 kNm *i.e.* the torsional capacity only marginally exceeded the torsional moment. In any event, only a maximum eccentricity of 5.7 mm in the loading was permissible before the torsional capacity of the section would be exceeded.

135 One could also use section 5.6 as a benchmark for comparison by relating the shear buckling strength under section 5.4.3 of BS 5950-5 to the average shear stress derived from section 5.6.1 (*i.e.* 3 % of the maximum force in the compression flange) and determine whether the average shear stress exceeded the shear buckling strength, as another means of checking the torsional capacity of a member. As I had noted, both JTC's expert and SEC's expert agreed that section 5.6 was not complied with.

136 It did not appear to me that Wyatt paid any special attention to the consequences of having only a three-sided member (and thus an "*open section*") at the ends, even though section 5.9 specifically highlighted the effects of torsion on open sections. Bearing in mind the minimal allowance of eccentricity in loading allowed before the torsional capacity of the stiffeners would be exceeded, Wyatt failed to heed section 5.9 of BS 5950-5 in his design.

137 SEC submitted that the concept of "torsion" had no application at all in the present as-built condition on site whereby the horizontal steel stiffeners had been sandwiched between the brick walls above and below them (*i.e.* packing) and were therefore restrained from "twisting" by the brick walls. I did not agree as the primary assumption in that submission was that the brick walls above and below were of the same height and had both bowed in the same direction to the same extent. However, where the brick walls above and below were not of the same height and had not bowed to the same extent, or where the brick walls above and below had bowed in the opposite direction into an "S" or "Z" shape, then there would be twisting and torsional effects induced in the horizontal stiffeners. The torsional strength of the stiffeners, including their ability to withstand side-sway deformation, would then become an important consideration. It would not be merely "*an academic exercise*", as described by Mr Jones.

iv. Effects of warping torsion increasing the torsional capacity

138 Warping torsion was a rather technical area which was not conceptually easy to comprehend. Nonetheless, after examining the opinions of the two experts, I preferred the analysis of Dr Ting over that of Jones that the end connections of the horizontal steel stiffeners as designed and built did not provide the necessary restraint and rigidity for warping effects/stresses to materialise sufficiently to increase to any extent the torsional capacity of the horizontal stiffeners in the manner described by Jones. Indeed, as Jones himself admitted:

> To have restraint for warping, you would have to have a very rigid connection, fully welded; and also all the plates would have to be of sufficient strength to transmit the warping stresses.

139 The welds prescribed for the end connections were, however:

i. 10 mm long at 10 mm centre to centre for the webs and full length weld at the top flange at the end connections if the horizontal stiffener was sitting on more than 35 mm of the support stub - *i.e*. the gap was 40 mm or less; and

ii. full length weld on the top flange as well as both webs if the horizontal stiffener was sitting on less than 35 mm of the support stub - *i.e*. the gap was more than 40 mm.

140 The prescribed welding at the end connections clearly did not meet Jones' requirement of a "very rigid connection, fully welded". The welding at the connections was in fact designed only to take the shear coming in from vertical and lateral loads. I accepted JTC's contention that the same welds could not be relied on to also bear the shear arising from torsional loads. This was *a fortiori* in the as-built scenario, where the welding design was not followed, and one-sided and irregular welding was observed.

141 Without restraint for warping at the end connections, the member could not be in a state of non-uniform / warping torsion. Accordingly, I found that SEC's suggestion that consideration should be given to the "*contribution to torsional resistance provided by warping torsion*" was misplaced.

v. Section 6.2 of BS 5950-5: Flexural Buckling

142 Section 6.2 related to flexural buckling and provided, *inter alia*, that:

> 6.2.1 Effective lengths
>
> The effective length of a member in compression should be established in accordance with Table 9 or on the basis of good engineering practice.
>
> 6.2.2 Maximum slenderness
>
> The slenderness ratio should be taken as the effective length, L_E, divided by the radius of gyration about the relevant axis, r; except as given in 6.2.5 for back-to-back members.
>
> The maximum values of the slenderness ratio L_E /r should not exceed the following:
>
> for members resisting loads other than wind loads: 180 …

143 JTC's position was that the maximum slenderness for the horizontal stiffeners, H1-250 and H1-300, exceeded the limit specified in section 6.2.2 of BS 5950-5.

144 Dr Chiew, whose main areas of research were in fatigue and fracture of steel structures, structural stability, light gauge steel structures, tubular structures as well as steel and composite construction, also agreed with JTC on this. Dr Chiew noted at page 2 of his test report on the steel stiffeners that:

> Except for H1-150 where web crushing controls, both H1-250 and H1-300 web maximum slenderness exceeds the limit of 180 specified in Clause 6.2.2 of BS 5950 Part 5. Elastic deformation is likely to occur in such high slenderness situations.

145 SEC however disputed those comments of its own witness, Dr Chiew. Firstly, SEC's expert, Jones, contended that a section 6 check was not required for the horizontal stiffeners. Secondly, Jones contended that the effective length L_E that should be used in calculating the slenderness ratio under section 6.2.2 should be 1.0L and not 1.2L.

a. Applicability of Section 6.2.2

146 Wyatt did not perform any calculations under section 6 of BS 5950-5 in respect of the horizontal stiffeners when he was designing the stiffeners. His contention was that as long as he satisfied "*the requirements for slenderness of a web, I've satisfied the code. I don't believe any designer would go to the compression part of the code [i.e. section 6] in trying to evaluate what he has to provide for a web*".

147 Jones also took the position that when checking the design of a beam in a spanning case, he would have done a section 6 check, specifically sections 6.2 and 6.4, for the vertical stiffeners but not for the horizontal stiffeners.

148 Dr Chiew, however, considered that a web buckling check should be done in "*standard design of beams*".

Q.	If a lintel or a steel member transfers the weight of the wall above it to the weight of the wall below – in other words, it acts more like packing; it will not be functioning as a beam. Would you agree?
A.	This is where, although we designed it as a beam, but there will be occasions -- even when we have designed as a beam, we also need to consider this packing or this compression, because there will be instances where there is actually loads coming from the top and bottom as well. So although we designed beams as such, that you transfer beams -- the loads to the supports, but we also have to consider this -- this mode of failure in beam design. We actually call it web bearing or web buckling, you see.
Q.	And it would be prudent to consider both when designing a beam?
A.	Yes. In standard beam design, you need to consider this as well.

149 I accepted that Dr Chiew's approach (which was also Dr Ting's) was the correct one.

150 Firstly, Jones did not dispute that a section 6 check was relevant in a packing scenario and did such a calculation in respect of the horizontal stiffeners (although he maintained that the calculations were undertaken to try and explain why the stiffeners experienced distortion on site and would not be part of a normal design process).

151 Jones had initially contended that section 6.2.2 of BS 5950-5 was for a member overall, and not for one plate in a built-up section *i.e*. part of a member, which was covered by section 4 of BS 5950-5. He agreed however that even if a designer had complied with section 4.2 of BS 5950-5, which was a purely dimensional check for "*maximum width to thickness ratios*", he should still go to section 6.2 for calculation of capacity.

152 When further clarification was sought, Jones conceded that in the packing scenario, the web would be treated as a "member" (not part of a member).

COURT:	I thought, when we look at page 299, which is clause 6, we are talking about a member. Why are we suddenly talking about a different (l), which is the packing? I thought we were looking at the whole member, the length.
A.	When you're looking at the vertical stiffener, you're looking at the --
COURT:	Whole length.
A.	-- whole length, yeah.
	We're now looking at a horizontal stiffener with a packing load going from top to bottom. The packing load goes through the webs of the horizontal member.
COURT:	So if you are evaluating the horizontal member as a packing member, you are now going to treat that element within that box to be the member itself, and then treat the length as the -- the web length as the length for table 9 at page 299?
A.	Yes, your Honour.
COURT:	That is a fair analogy, is it?
A.	That's a fair analogy, yes, your Honour.
A.	So you won't dispute that for packing purposes, I treat the web as the member?
A.	Yes, your Honour.

153 Notwithstanding Jones' objections to applying section 6.2 to the web, I accepted the approach

of Dr Chiew and Dr Ting (JTC's expert) that in a packing scenario, the correct approach was to treat the web as a member.

154 In fact, Wyatt himself had done a section 6 check of the horizontal stiffeners in 2004, after the defects had manifested on site, noting that it was the "*more rigorous calculation*".

155 Even if Jones' and Wyatt's position that a section 6 check was not required when designing a beam in the spanning case were to be accepted, Wyatt was however well aware that the stiffeners that he was designing were meant to stiffen the in-fill brick walls, *i.e.*, that there would be a packing load going through the webs of the horizontal stiffener. Wyatt clearly knew that below each horizontal stiffener was a wall supporting the stiffener from beneath, and hence there would be inevitably be some packing loads. Under the circumstances, such a check should have been done to ensure that the stiffeners would be fit for their intended purpose. This, Wyatt had failed to do.

156 Thus, in my judgment SEC should be liable for breach of its obligations to JTC in its design of the steel stiffeners if the stiffeners failed a section 6.2.2 check, even if such a check would only have been required in a packing scenario.

b. Whether the maximum slenderness of the H1-250 and H1-300 exceeded the limit specified in section 6.2.2 of BS 5950-5

157 Under section 6.2.2 of BS 5950-5, the maximum value of the slenderness ratio, L_E/r (where L_E stands for effective length and r stands for the radius of gyration about the relevant axis) for members resisting loads other than wind loads (*i.e.* the horizontal steel stiffeners) should not exceed 180.

158 Table 9 of BS 5950-5, in turn, sets out the effective lengths for compression members. It provides, *inter alia*, the following:

Table 9- Effective Lengths, L_E, for compression members

Conditions of restraints at ends (in plane under consideration)	Effective Length
Effectively held in position at both ends but not restrained in direction	1.0L
Effectively held in position and restrained in direction at one end with the other end effectively restrained in direction but not held in position	1.2L

159 JTC's expert, Dr Ting, considered that an effective length of 1.2L should be used under Table 9. SEC's witness, Dr Chiew, agreed as well, noting at page 2 of his test report that:

> The effective length factor for web buckling calculations used should be at least 1.2, a more conservative value of 1.5 is recommended since the side-sway deformation is predominant.

160 On the stand, Dr Chiew explained that he had considered 1.2L as a reasonable value for

effective length as it allowed a bit of relative movement and so it was not entirely pinned at both ends. He further explained that 1.0L was for the condition where both ends were held in position, and they could not move left and right but they were free to rotate.

161 It was plain from the side-sway of the specimens observed in Dr Chiew's laboratory tests that the horizontal steel stiffeners were not held in position at both ends. Indeed, Dr Chiew was also concerned about rotation; hence his recommendation of a more conservative value of 1.5L for the effective length.

162 Notwithstanding the side-sway of the stiffener specimens observed in Dr Chiew's laboratory test, and Dr Chiew's own comments, counsel for SEC had initially suggested to Dr Ting that an effective length of 0.7L (for compression members "*effectively held in position and restrained in direction at both ends*") was appropriate. Not surprisingly, Dr Ting responded that "*0.7 to me is dreaming*".

163 When he took the stand, Jones then sought to put forward a more reasonable position, contending that the effective length was 1.0L, relying on the results of Dr Chiew's tests - that the web buckling load calculated for 1.0L was closest to the ultimate load during the test. In my view, an effective length of 1.0L was not supportable in the light of clear evidence of side-sway deformation observed in Dr Chiew's test. Indeed, side-sway was observed not only in respect of the H1-250 and H1-300 specimens (where the predominant mode of failure was web buckling) but in respect of the H1-150 specimen as well (where the predominant mode of failure was web crushing). Jones eventually conceded however that the effective length was probably somewhere between 1.0L and 1.2L.

164 In any event, in the light of Jones' admission that the web was to be treated as a member under section 6.2.2 in the packing scenario, it was irrelevant in my view whether or not 1.0L or 1.2L was adopted as the effective length as the maximum slenderness ratio of the web exceeded 180 even if an effective length of 1.0L was adopted.

165 In respect of H1-250, applying an effective length of 1.2L, Dr Ting had derived a slenderness ratio of 494. Even if an effective length of 1.0L was adopted, the slenderness ratio would be 412. This still exceeded the maximum ratio of 180 prescribed by section 6.2.2 of BS 5950-5.

166 Similarly, for the H1-300, adopting an effective length of 1.2L, Dr Ting had derived a slenderness ratio of 479. Even if an effective length of 1.0L was applied, the slenderness ratio of the web would be 399, which far exceeded the maximum ratio of 180 prescribed.

167 In my judgment, the design of the steel stiffeners for H1-250 and H1-300 had failed to comply with section 6.2.2 of BS 5950-5 and this constituted yet another breach of SEC's obligations.

c. Whether the maximum slenderness for H1-250 and H1-300 would still be non-compliant with code requirements if section 4.2 of BS 5950-5 was applied

168 Jones contended that the relevant slenderness limit for individual plates in a built-up section was section 4.2 of BS 5950-5, which stated:

> 4.2 Maximum width to thickness ratios
>
> The maximum ratios of element flat width, b, to thickness, t, which are covered by the design procedures given in this part of BS 5950 are as follows, for compression elements.

a) Stiffened elements having one longitudinal edge connected to a flange or web element, the other stiffened by:

simple lip (see Figure 2)

60

any other type of stiffener conforming to 4.6

90

b) Stiffened elements with both longitudinal edges

connected to other stiffened element

500

c) Unstiffened compression elements

60

Note: Unstiffened compression elements that have width to thickness ratios exceeding approximately 30 and stiffened compression elements that have width to thickness ratios exceeding approximately 250 are likely to develop noticeable deformations at the full working load, without affecting the ability of the member to carry this load.

169 Jones further contended that the applicable ratio in this case was 500, *i.e*. Jones considered the web to be a "*stiffened [element] with both longitudinal edges connected to other stiffened elements*".

170 Dr Ting did not dispute that a section 4.2 check could be done on the web in addition to a section 6.2 check (although he considered it pointless to proceed with a section 4.2 check if the web failed to comply with section 6.2).

171 However, Dr Ting disagreed that the applicable ratio under section 4.2 was 500. This ratio was only applicable if each of the elements (*i.e*. the two webs and the top and bottom flanges) in the built-up section was a stiffened element. In this case, the flanges of the steel stiffeners, which were simply flat plates (unlike the webs which had lips on both ends), were not themselves stiffened elements.

172 In the event, the slenderness ratio that could be applied to the steel stiffeners under section 4.2 of BS 5950-5 was, at best, a ratio of 90 (if not 60).

173 Jones had computed the maximum width to thickness ratio to be 123, for H1-250 and 119, for H1-300, *i.e*. exceeding the ratio of 90, the best ratio applicable in this case. In other words, the H1-250 and H1-300 steel stiffeners would only be compliant with section 4.2 if the applicable ratio was 500.

174 Even if I were to accept SEC's contention that the applicable slenderness ratio under section 4.2 was 500, this only applied to the box sections with 4 plates. Jones agreed that in the case of a U-section, only the bottom flange was stiffened, not the other 2 elements of the "U" - *i.e*.

the webs.

175 The inescapable conclusion then was that at the end sections of the horizontal stiffener, where the bottom plate was removed and there were just 3 plates (*i.e*. an inverse U-section), the webs would be unstiffened elements and these unstiffened webs at the end sections would not comply with the slenderness ratios prescribed under section 4.2 of BS 5950-5.

176 For the reasons stated, I concluded that the design of the steel stiffeners failed to comply with the applicable provisions of BS 5950-5 in several aspects. That was in and of itself a breach of note 2 of the Standard Specification for Construction Works. I also accepted the submission of JTC's counsel that this represented a failure by SEC to use all reasonable skill, care and diligence in the design of the steel stiffeners, for which SEC should be found liable to JTC. It was readily apparent from the defects found on the site and the side-sway observed in Dr Chiew's test that the slenderness of webs had an adverse effect on the performance of the steel stiffeners.

B. Construction and Workmanship

177 JTC contended that SEC failed to construct the steel stiffeners using all reasonable skill, care and diligence and in a good and workmanlike manner in breach of its obligations to JTC. The following evidence was adduced before me.

i. Defects observed after stiffeners were exposed

a. Incomplete stiffeners

178 When the horizontal stiffener at the external face of the staircase A brick wall at the 2nd production floor of unit #05-11 was exposed, it was seen that the vertical face of the stiffener only extended midway through the wall, with the remaining area filled in with bricks.

b. No stub to connect the steel stiffeners

179 Instances of the end of the steel stiffeners being partially supported by the support stub had been noted by Setsco Services Pte Ltd ("Setsco") in its inspection of the Development: for instance, see photograph no. C3-7 at page 367 of Setsco's Joint Affidavit.

180 When Dr Ting and McGowan from Babtie Asia attended at a joint inspection of unit #03-12 (which was under repair), they observed that there was no support stub at the connection between the H-150 horizontal stiffener (left span) and the V1-50 stiffener at Staircase B on the 1st production level.

181 Dr Ting elaborated further on the stand, with reference to pages 37 and 38 of P-35 (photos taken by Dr Ting at the inspection) that he could insert a measuring tape into the gap between the horizontal and vertical stiffener, for a length of over 90 mm. This evidenced that the horizontal stiffener was not resting on a stub.

182 Jones could only say in response to such poor workmanship that "*my view is that the* ***contractor took a calculated risk that he could get away with it****, on the basis that it wasn't really a wall of great significance in terms of stability; it's a staircase wall with limited application for load carrying*" (emphasis added).

183 Evidently, the risk did not pay off as repair works had to be carried out at the location and the

stiffener was, literally, exposed!

c. Uneven support

184 There was also photographic evidence of mortar joints beneath a horizontal steel stiffener not being fully filled (Agreed Bundle of Documents at 7602 and 7603, showing photographs of the staircase A landing at the 2nd production level of #05-11). The horizontal stiffener was therefore not evenly supported.

185 In an email from Wyatt to Ng Wee Beng dated 19 October 2005, Wyatt expressed strong concerns on the uneven support to the stiffeners, stating:

> My feeling is that uneven support to the Stiflex is a significant factor. NTU testing with uneven support may confirm that Stiflex capacity under these conditions is much closer to the weight of brickwork over. We have seen uneven support for a number of the problem Stiflex recently.

186 Wyatt also confirmed on the stand that the capacity of the stiffener would be affected if there was non-uniform support.

d. Welding

187 The welding of the stiffeners was also found to be far from satisfactory.

1. WELDING BETWEEN FLANGE AND WEB

188 The original welding detail for joining the web plates with the flange plates was spot welds of 4 mm at 40 mm centre to centre. In March 1999, Wyatt provided an "*alternate approach*" of 4 mm spot welds at 300 mm centre to centre and 10 mm length weld at 50 mm spacing.

189 Setsco had conducted a visual inspection of a 500 mm steel stiffener section obtained from the Development. Setsco's report indicated that of the 43 spot welds on the steel stiffener, only 1 had complete fusion, 20 had penetration and no fusion, and 22 had no penetration and no fusion. For this sample, the welding was of poor quality in my view. While I acknowledged that the sample size was small and the welding quality for the sample might not be representative or indicative of the general welding standard for the steel stiffeners throughout the Development, I could only say that the results of the welding inspection of this one sample did not give me much confidence on the general quality of the welding for the steel stiffeners used in the Development.

190 When these results were shown to Sim, he confirmed that he would not consider this kind of welding acceptable as he considered complete fusion an important criteria. Needless to say, without complete fusion, the strength of the welds would be severely compromised resulting in a weakening of the capacity of the steel stiffeners to carry their designed loads.

191 Wyatt similarly agreed that based on these results, the welding carried out was not satisfactory.

192 When WP Brown carried out investigations on the defects observed at the stiffener areas at the two toilets in the SS 3000 series factories, Wyatt also noted in a report dated 16 August 2004 that:

> … the weld between the side plate and top and bottom plates differed from the nominated detail.

> The provided weld considered of 10 mm nominal length of fillet weld at approximately 200 mm spacing. This appeared to be the temporary welding used during assembly, indicating that this component had most likely been issued to site without the final full welding being completed.

193 The poor welding in my view had a significant effect on the buckling capacity of the H1-250 stiffener, reducing the factor of safety to less than 1. Wyatt commented as such:

> For the assumptions adopted to estimate the effects of the reduced welding provided, it has been found that the resistance to buckling is inadequate.
>
> If the adopted rectification procedure involves retaining the existing Stiflex, then it would be necessary to incorporate upgrading of the welding between the steel plates.
>
> (Emphasis added)

194 Indeed, Wyatt confirmed on the stand that non-compliant welding affected the performance of the stiffeners.

Q.	We have in large part been focusing in our discussions on the design, but would you agree, Mr Wyatt, as a general statement, that if there is a departure from your design in the form of welding that is less than satisfactory, that has an impact on the performance of the Stiflex?
A.	In packing?
Q.	In packing.
A:	Yes. Yes.

195 It should be highlighted in this regard that Jones confirmed that in Wyatt's checks on weld capacity in respect of a H1-300 steel stiffener at Elevation 4 of a SS 2000 unit (see AB 4182 and 4183, where he had derived a critical weld shear of 0.034 kN/mm and a total capacity of 0.033 kN/mm), Wyatt had calculated the capacity of the spot welds assuming they had complete penetration and fusion. While Jones sought to defend Wyatt's calculations as being "conservative", he agreed however that this would be the approach he would take when designing in advance of construction.

196 In any event, the inescapable conclusion was that Wyatt's design was exacting on the fabricator in that each and every weld was counted on to add up to sufficient weld capacity. Wyatt tried to defend his calculation, arguing that the code provided acceptable minimum factors of safety and thus, there should be no problem using the full capacity permitted under the code. Indeed, as Wyatt himself admitted, "*even if you have a design that complies with the code, if you have a shortfall in workmanship, that can lead to a shortfall in overall performance*".

2. WELDING AT END CONNECTIONS

197 The welding of the stiffeners was also found to be non-compliant with the welding stipulated by Wyatt.

198 Wyatt had prescribed the following welds:

i. Welding at 10 mm long at 10 mm centre to centre for the webs and full length weld at the top flange at the end connections if the horizontal stiffener sat on more than 35 mm of the support stub - *i.e*. the gap was 40 mm or less; and

ii. full length weld on the top flange as well as both webs if the horizontal stiffener sat on less than 35 mm of the support stub - *i.e*. the gap was more than 40 mm.

199 Setsco had carried out break-out inspection of the end connections of the steel stiffeners at units #03-21, #03-27 and #05-29 of the Development. Of the 12 breakout points on the 6 stiffeners, 7 points were found to be without welding and 4 of the 6 stiffeners were found to have no welding on at least one plate at the end connection.

200 Dr Ting considered that the welding on one side of the steel stiffeners would introduce a torsional moment.

201 Indeed, on this issue of non-compliant welding at the end connections, Jones admitted that he would not have been happy:

Q. One perhaps more glaring instance is that the welding at the end connections does not seem to have followed the nominated detail at all. To be specific, the contractor seems to have welded one side, not the other side, whereas Mr Wyatt's nominated detail was intermittent welding on both sides, welding at the top, and so on; it was thought through and calculated based on what he expected in terms of shear hand capacity and so on, and from what we can see, the contractor seems to have more or less disregarded it and just done whatever he wanted to do. If you were a designer, surely that would be unacceptable to you?

A. I wouldn't be very happy, but then it would depend on how much fat I've left in my design to allow for the fact you've got a Singaporean contractor on site building it…

202 It should be highlighted that the welding at the webs of the end connections were observed to be "*irregular continuous weldment*".

3. WELDING TO JOIN TWO WEB PLATES

203 Setsco also conducted a reduced section tensile test on the joints to connect two steel plates to form the flange plate of a stiffener box-section.

204 The results of the test indicated that the tensile strength of the parent material was in excess

of 500 N/mm^2 whereas the tensile strength of the welded plate was only 316.3 N/mm^2. In other words, the weld joining the two plates was found to be weaker than the parent material. In fact, the weld was weaker than the material strength of the steel assumed in the design - 365 N/mm^2.

205 Jones suggested that the weld that was tested was a 4 mm spot weld and that the "correct" tensile strength of the weld based on this weld length should thus be approximately 790 N/mm^2, a value that was off the charts in the British Standards. By Table 1 of BS EN 499 (1995), the highest value for tensile strength of a weld was 720 N/mm^2 with grade E50 electrodes used. Wyatt's recollection was that E43 electrodes, a lower grade of electrodes than E50, were used in the present case. I thus rejected the suggestion of Jones that the tensile strength of the weld as tested could have reached the incredibly high tensile strength of 790 N/mm^2 because of the wrong assumption that the weld present at the flange-to-flange joint was actually 3 spot welds, which was contrary to the evidence. Mr Cheng Kwang Meng, the representative from Setsco who had supervised the reduced section tensile tests, confirmed on the stand however that the weld was a continuous weld across the two flanges that were joined together, and not spot welds. Wyatt, the designer, testified that although he had not specified the welding detail for the flange-to-flange connections, he would have expected it to be a full butt weld (*i.e*. not spot welds).

206 In the event, the weld failed what Jones himself described as "*the fundamental premise of welding*" as the weld was weaker than the parent material.

207 Wyatt also commented that the welds used to join the two plates did not look like the full penetration butt welds which he would have expected the fabricator to do.

208 In light of the evidence of poor workmanship/construction highlighted above, I found that SEC had failed to construct the steel stiffeners using all reasonable skill, care and diligence and in a good and workmanlike manner, in breach of its obligations to JTC.

C. Stiffeners were Unfit

i. Steel stiffeners caused defects

209 As stated above, for the stiffener system to ensure wall stability by preventing the lateral buckling of the brick wall, it necessarily had to be stiffer than the brick wall for it to be useful as a wall stiffener.

210 However, it was clear that in the on-site situation, the steel stiffener system had not performed satisfactorily. Buckling of the plaster (with horizontal cracks at the centre line of the buckled portion of the plaster) was observed both on the external and the internal partition walls. The buckled areas coincided with the positions of the steel stiffeners. When the buckled plasters were removed, the webs of the steel stiffeners appeared to be distorted; further, in a number of locations, the brick walls above and below the steel stiffener were noted not to be in line (*i.e*. out of plumb with one another).

211 The parties' respective experts formulated various theories as to the main cause of these defects observed. However, the primary theories favoured by each expert were as follows:

(a) JTC's expert, Dr Ting, opined that the steel stiffeners failed primarily on account of their low lateral stiffness under lateral load (*i.e.* wind load); of course, the other factors (such as vertical loads) aggravated the situation;

(b) SEC's expert, Jones, opined that the steel stiffeners were the "weakest link" under compressive loads (primarily the creep and shrinkage of the structure, acting in tandem with the expansion of the bricks), and the defects observed were caused by such compressive loads.

212 However, irrespective whether Dr Ting's opinion or Jones' opinion was the correct one, the steel stiffeners had failed (whether on account of lateral loads or compressive loads) and were therefore unfit for use. I agreed with the submission of counsel for JTC that on this alone, JTC's claim would succeed.

a. How the steel stiffeners caused the defects on site: Dr Ting’s opinion

213 Dr Ting's opinion was that the lateral load was the most significant cause of the failure of the steel stiffeners. He explained that firstly, the lateral stiffness of the vertical steel stiffeners was so low that their lateral deflections under lateral load were already very large. To make things worse, the horizontal steel stiffeners that were depending on the vertical steel stiffeners for support would also be displaced laterally. These lateral displacements resulting from the deflections of the vertical steel stiffeners would be additional to those lateral deflections of the horizontal steel stiffeners, thus aggravating the situation. With the weight of the brick wall acting on the laterally-displaced horizontal steel stiffeners, the slenderness of the webs would result in the horizontal steel stiffeners being prone to side-sway and even more lateral displacement. These lateral displacements and side-sway of the horizontal steel stiffeners then resulted in the rotation and/or twisting of the horizontal steel stiffeners along their lengths. As such, the weakness of the horizontal steel stiffeners in torsion further aggravated their deformation.

214 The following was rendered to explain why the defects manifested themselves after some time. An initial gust of wind would result in a Z-shape deformation of the wall as the horizontal steel stiffeners would deflect laterally, and the weight of the wall acting on the slender webs of the horizontal steel stiffeners would cause the members to go into a side-sway mode of deformation. After the gust of wind, the deformed webs would not be able to return to their original vertical position as each of the horizontal steel stiffeners had to support a brick wall above them. The deformation would also result in a twisting load on the horizontal steel stiffeners.

215 Another gust in the same direction would further aggravate this deformation as the vertical stiffness of the horizontal steel stiffener would be reduced due to the side-sway of the two webs. But a gust in the opposite direction would tend to reduce the deformation. As wind loads of varying magnitudes and directions occurred daily, there would be a continuous lateral displacement of the steel stiffeners, especially when the deformed webs were unlikely return to their original vertical position due to the continuously imposed vertical loads from the weight of the brick wall above them.

216 These cumulative deformations were small for small wind gusts. The plaster would crack either when the gusts were stronger or when the cumulative deformations were more than the plaster could take. This explained why the defects took some time to manifest themselves.

217 After careful consideration I accepted Dr Ting's opinion (summarised in the above paragraphs) of what caused the defects observed on site. I now analyse the following causation factors in greater detail:

(a) low lateral stiffness under lateral load;

(b) slenderness of the webs resulting in the steel stiffeners being prone to side-sway;

(c) low torsional capacity of the steel stiffeners;

(d) poor construction of the steel stiffeners.

1. STIFFENERS PRONE TO EXCESSIVE DEFLECTION UNDER LATERAL LOAD

218 Dr Ting considered that the steel stiffeners were prone to excessive deflection under lateral load, as they had low lateral stiffness.

219 Clearly, lateral load in this case was brought about primarily by wind, which was a real phenomenon present on site. As such, Wyatt, as the designer of the steel stiffeners, ought to have designed steel stiffeners which were capable of bearing such loads, without the stiffeners themselves deforming or deflecting excessively (thereby causing damage to the wall finishes).

(I) WIND LOAD WAS CONSIDERED

220 Wyatt had undertaken his design calculations on the basis of a wind load of 0.5 kN/m^2 for the external walls and 0.2 kN/m^2 for the internal walls.

221 In this regard, Jones also used a wind load of 0.5 kN/m^2 in his calculations, which he considered would translate to a wind speed of between 32 to 35 m/s at 50 metres above ground level.

222 Most significantly, Jones himself would design for wind load if he was the designer, and in this regard, he considered that a wind load of 0.5 kN/m^2 was reasonable. I noted that Jones in his 1st report at paragraph 6.6.2 had in fact acknowledged that the horizontal stiffeners acted primarily as members to carry the lateral wind load as follows:

> The Dyntek horizontal stiffeners, or lintels, act **primarily as members to carry lateral wind load** as the 100 mm thick brick infill walls at the full panel size between primary structural elements is unable to safely carry the Building Regulation specified wind load without being divided into smaller panels or elevation. (Emphasis added)

223 However, despite some consideration of wind load in the design of the steel stiffeners, the steel stiffeners still left much to be desired in that respect in any event.

(II) EXCESSIVE DEFLECTIONS UNDER WIND LOAD

224 The different opinions of JTC's expert, Dr Ting, and SEC's expert, Jones, as well as that of Wyatt (the designer of the steel stiffeners), in respect of deflection limits, have been set out above, at [83] onwards.

225 Also, as explained above, the lateral deflections under wind load of the steel stiffeners in brick wall panels found on site were still unacceptable, as calculated by Dr Ting (in P-38) and by Jones (in D-42).

226 For ease of reference, the table of comparison of steel vertical stiffener lateral deflections on page 1 of D-42 is reproduced herein as follows (with the lateral deflections which exceeded Jones' requirements highlighted in bold and underlined).

STIFFENER LOCATION	JONES	DR TING
#03-12 ELEVATION 1 EXTERNAL WALL (V1-100)	**<u>54.4 mm (H/97)</u>**	**<u>58.9 mm (H/90)</u>**
#03-12 INTERNAL WALL (V1-50)	19.2 mm*1 (H / 231)	**<u>47.1 mm (H/119)</u>**
JONES P335 REVISED EXTERNAL WALL (V1-200)	**<u>STEEL 43.9mm (44.3mm)*4 (H/119) (H/118)</u>**	**<u>41.0 mm (H/128)</u>**
	COMPOSITE 3.5 mm (H / 1496)	
WYATT PAGE 26 EXTERNAL WALL (V1-200)	**<u>38.9 mm*3 (H/132)</u>**	**<u>36.8 mm (H/143)</u>**
7m x 10m*2 EXTERNAL WALL (V1-200)	**<u>PIN 155.6 mm (H/45)</u>**	**<u>147.2 mm (H/48)</u>**
	<u>PARTIAL RESTRAINT 113.5 mm (H/62)</u>	

**1 - ACTUAL HEIGHT USED OF 4.45 m c.f. TING 5.6 m*

**2 - NOT AN AS-BUILT OR AS-DESIGNED CASE*

**3 - HORIZONTAL SPANS INCORRECT*

**4 - CALCULATED FOR I = 2.35 X 10*^{6} *AND I = 2.33 X 10*^{6} *FOR V1-200.*

227 As such, it was apparent from D-42 that of all the 4 locations actually found on site, Dr Ting calculated the lateral deflections of the vertical steel stiffeners to be such that they would not comply with Jones' limits for lateral deflections. See also P-38.

228 On Jones' own calculations, almost all the lateral deflections would have been unacceptable to Jones. Only the lateral deflections of the vertical steel stiffeners at the #03-12 internal wall and using

the "composite analysis" for page 335 of Jones' 1st AEIC, would produce acceptable deflection ratios (on Jones' standards). The reason why Jones' calculation for the deflection of 3.5 mm for the “composite analysis” was less than one tenth of the deflection of 41mm computed by Dr Ting's calculation is set out below.

229 If the deflection ratio was to be calculated for the external wall at Jones' 1st AEIC, page 335, on the basis of the vertical steel stiffener alone, it would not meet Jones' requirements. What D-42 and D-43 aimed to demonstrate was that when a 1 m-strip of brick wall on either side of the vertical steel stiffener was taken together with the vertical steel stiffener, a "*great stiffening effect on the steel stiffener itself*" would result because of the additional stiffness provided by the brickwork attached to the vertical steel stiffener by the shear transfer dowels, and the render, and by assuming that they acted compositely.

230 I accepted JTC’s argument that in this regard, if SEC had to rely on the brickwork on either side of the vertical steel stiffener to provide a "*great stiffening effect on the steel stiffener itself*" such that the calculated lateral deflection would be reduced from 43.9 mm to 3.5 mm as a result, this must in itself be an admission that the steel stiffeners were not fit for their intended purpose. On the evidence and on the basis of the detailed submissions by counsel for JTC on this point, I was not satisfied that the brickwork could provide the alleged “*great stiffening effect*”. I rejected Jones’ explanation on how such a “*great stiffening effect*” could be realised in the actual construction of the wall and the stiffeners, particularly if the brick wall and plasters were cracked. Stiffness of cracked plaster and brickwork would be close to zero. It was also clear from the available photographs and other evidence, that the plaster/render was not properly bonded to the steel stiffeners and would not thereby confer any "*compositeness*" on the steel stiffeners to increase their rigidity.

231 As such, at all four on-site locations, there would be lateral deflections suffered by the steel stiffeners, with unacceptable deflection ratios, even by the (less stringent) standards Jones would apply. This was because by these deflections, the plaster/render would already have cracked. This was borne out by what was observed on site. The irresistible conclusion was that the steel stiffeners were not fit for their intended purpose, and on this basis alone, I found that SEC was liable to JTC.

(III) WIND SPEED

232 Dr Ting had calculated these deflections on the basis of 0.5 kN/m^2 wind load with a return period of 50 years. For the steel stiffener system, in both the 7 x 10 m (hypothetical) case as well as the external wall shown at page 335 of Jones' 1st AEIC (actual case), the lateral deflections calculated by Dr Ting for the corresponding periods of 1.01, 2, 5, and 10 years would not comply with BS5950 Part 5. Likewise for the steel stiffener system at the external wall shown at page 26 of Wyatt's AEIC, the relevant period was just 1.013 years.

233 In any event, even by Jones' own "requirements", the steel stiffeners did not comply on the basis of excess lateral deflection, and Jones himself would not be happy with that because at those deflections, the plaster / render would already have cracked. In other words, the lateral deflections due to wind load would not be acceptable to either party's expert.

(IV) CONSEQUENCES

234 Looking at the matter from another angle, Jones also agreed that one would not need to have massive wind loads for the stiffeners to be pushed out. Further, it would not follow from the fact that the wind loads were transient loads that the horizontal deflections would not occur or would not be permanent if they occurred. If the stiffener was pushed out by the transient wind load, the stiffener

would not be able to move back to its original position due to the vertical load of the wall above pressing down on it.

> COURT: Very early on you're saying that the design would use that, so you're saying that in actual fact the wind won't blow so strongly.
>
> A. Correct. I'm saying that the wind loads as calculated were conservative.
>
> COURT: ***So what would be the kind of wind loads we are talking about that would not be conservative? So then I will look at what is the reasonable deflection due to that reasonable wind load, then I will see whether or not it will drive it beyond the popping point. Because the instability is such that, if I move it such, it would immediately have the disequilibrium and you will reach that stage because your pressure is there all time. I just need a very soft push and it will automatically assume that shape, so I don't need very high wind loads to do that -- would I be correct -- because it's unstable***?
>
> A. **That's reasonable, yes**.
>
> COURT: Well, then I don't need all this massive wind load. I just need to push and it that's it, you can't pull it back and it would remain down there permanently deflected for the simple reason it must achieve that 2.7?
>
> A. ***Yes***.
>
> COURT: ***And to force something that's suspended back in to the (unclear) wall is nigh impossible***, just like building a bridge, you know why you build it concave-shape, because the ability to (unclear) pressure, so the moment so I push it out and [to] pull it back, it's going to be damn tough -- correct?
>
> A. **Yes, your Honour**.
>
> (Emphasis added)

235 As such, a cause of the defects observed on site was that the steel stiffeners were unfit and unable to carry wind load satisfactorily, such that at the wind speeds experienced at the Development, the steel stiffeners would deflect by so much that the plaster / render on the brick walls would have cracked.

236 And since it was almost impossible for the steel stiffeners to then recover completely to their original position, there would be some permanent lateral displacement of the steel stiffeners, consistent with what was observed on site. This was also consistent with the fact that the defects manifested themselves over a period of time.

2. HORIZONTAL STEEL STIFFENERS ALSO HAD THE PROPENSITY TO SIDE-SWAY

237 According to Dr Ting, the horizontal steel stiffeners would have an innate propensity for

displaying side-sway deformation as a result of the slenderness of the webs (in particular, for the H1-250 and H1-300 horizontal steel stiffeners).

238 Thus, with the entire horizontal steel stiffener deflecting significantly under lateral load, the horizontal steel stiffener would also tend to side-sway, with the following factors playing their part to achieve that:

(a)	most horizontal steel stiffeners were not at the exact mid-height of the entire brick wall (*i.e.* between the floor slab above and the floor slab below);
(b)	the weight of the brick wall would act on the horizontal steel stiffener in the vertical direction, but coupled with the above factor, the horizontal steel stiffener would experience eccentric vertical loads acting on it;
(c)	the two flanges of the horizontal steel stiffener would tend to displace horizontally, relative to each other, under the eccentric vertical loads;
(d)	the horizontal steel stiffener (with its innate propensity to side-sway) would exhibit side-sway deformation;
(e)	with the horizontal steel stiffener experiencing some side-sway deformation, its stiffness under vertical loads would be further reduced, and hence, it would become even more likely to side-sway under the next gust of wind (and under the same vertical load from the wall).

239 Furthermore, the occurrence of the side-sway deformation of the horizontal steel stiffener would also introduce further eccentricity into the steel section, and correspondingly, induce torsional moments in the horizontal steel stiffeners.

(I) DR CHIEW'S TEST RESULTS

240 The results of the laboratory tests and investigations on the behaviour of the horizontal steel stiffeners under load conducted by Dr Chiew provided ample evidence of the steel stiffeners' propensity to side-sway.

241 Dr Chiew tested certain H1-150, H1-200 and H1-300 specimens to failure. Dr Chiew observed (at page 143 of his AEIC) that the elastic side-sway deformations could be quite significant especially in the H1-250 and H1-300 specimens, noting:

> Except for H1-150 where web crushing controls, both H1-250 and H1-300 web maximum slenderness exceeds the limit of 180 specified in Clause 6.2.2 of BS5950 Part 5. Elastic deformation is likely to occur in such high slenderness situations.

242 Dr Chiew summarised his test results in Table 2 (“TEST RESULTS & ANALYSIS”) as follows:

Specimen	Ultimate load during test (kN)	Web crushing P_w (kN)	Web buckling		Remarks
			L_e	P_b (kN)	

H1-250(1)	41	56	1.0d	44	- Use as trial specimen before - Intermittent weld along lip gave way **under excessive sidesways**
			1.2d	31	
			1.5d	20	
			2.0d	11	
H1-250(2)	40	56	1.0d	44	- No weld failure - **Excessive sidesway**
			1.2d	31	
			1.5d	20	
			2.0d	11	
H1-300(1)	65	83	1.0d	59	- No weld failure - Not so much **sidesway**
			1.2d	41	
			1.5d	27	
			2.0d	15	
H1-300(2)	67	83	1.0d	59	- **Specimen collapse due to excessive sidesway**
			1.2d	41	
			1.5d	27	
			2.0d	15	
H1-150(1)	135	148	1.0d	410	- **Excessive sidesway** - **Whole assembly collapse with weld failure** - Permanent deformation with no elastic recovery
			1.2d	288	
			1.5d	187	
			2.0d	106	
H1-150(2)	170	148	1.0d	410	- No weld failure - Not so much **sidesway**
			1.2d	288	

			1.5d	187	
			2.0d	106	

(Emphasis added)

243 As such, Dr Chiew's tests showed that under compressive loads, the predominant mode of failure for the H1-250 and H1-300 horizontal steel stiffeners was that of web buckling, while for H1-150, the predominant mode of failure was that of web crushing. For both modes of failure, side-sway deformation was observed, with 4 out of 6 specimens exhibiting excessive side-sway, and at least 1 out of 6 specimens collapsing due to excessive side-sway. There were also on site pictures of horizontal steel stiffeners exhibiting side-sway deformation. Jones' view was that the distortion observed on site was that of side-sway as per Dr Chiew's test.

> *COURT*: ***But we have seen the top plates move laterally relative to the bottom plate****, so you have -- they call it failure --*
>
> *A*. ***It's what I call "side sway"***.
>
> (Emphasis added)

244 In fact, on site, such side-sway deformation of the stiffeners was commonly observed near the delamination and cracking of plaster. More specifically, Jones' evidence was that the form of distortion which he had seen on site was similar to the distortion which was seen in Dr Chiew's laboratory tests, which Jones would describe as a combination of local web crushing and side-sway.

245 Even in a pure compression case as tested in the laboratory, the elastic side-sway deformations were already significant for H1-250 and H1-300 horizontal steel stiffeners in particular. As the actual site conditions were even tougher, with wind load and eccentricity (due to the weight of the brick wall above acting on the laterally deflected horizontal steel stiffeners) added into the mix, instability was generated in the system when the flanges of the horizontal steel stiffeners moved relative to each other, causing the horizontal steel stiffener to attempt to rotate. I further noted that the loading beam used in Dr Chiew's laboratory tests to apply the loads on the horizontal steel stiffener was a "*big piece of steel*", which was much stiffer and did not bow, like the brick wall. The horizontal stiffener also rested on a large concrete block. This test set-up was thus far more rigid than the on-site conditions, where the steel stiffeners were installed with brick walls above and beneath them. Further, this rigidity had the effect of constraining side-sway of the steel stiffeners, which correspondingly increased the ultimate load they could withstand during the test. Hence, the test results for the purpose of establishing the carrying capacity of the stiffeners already had a bias in favour of a carrying capacity that was higher than it would otherwise have been in the as-built or on-site condition.

3. STIFFENERS HAD LOW TORSIONAL CAPACITY AND WERE PRONE TO TWISTING

246 On Dr Ting's account, when the steel stiffeners were subjected to lateral load, they would deflect laterally, and further, experience side-sway deformation. These effects would tend to induce a rotation or twist in the horizontal steel stiffeners as a result of its low torsional capacity.

247 This was borne out by observations of the defects on site. When the buckled plasters were removed, the webs of the steel stiffeners appeared to be distorted; further, in a number of locations, the brick walls above and below the steel stiffener were noted not to be in line (*i.e*. out of plumb with one another).

248 These manifestations indicated both lateral (*i.e*. out-of-plane) movement of the brick walls above and below the horizontal steel stiffeners (caused by the side-sway deformation of the horizontal steel stiffeners), as well as the twisting and/or rotation of the horizontal steel stiffeners.

(I) WAS THERE SOME ROTATION / TWIST / TORSION?

249 JTC's position was that rotation and/or twist of the horizontal steel stiffeners was found on site, consistent with the low torsional capacity of the horizontal steel stiffeners.

250 However, Jones was reluctant to even agree that there was rotation and/or twist of the horizontal steel stiffeners, and kept changing his position in this regard. Jones had initially taken the position that there was rotation of the steel stiffeners observed on site, around May 2007, being the date of his 1st report (contained in his 1st AEIC where he stated at [3.3] under section 3.1 on "CONDITION SURVEY" that:

> The general findings of the condition survey are representative of defects arising from the absence of horizontal compressible or soft joints between the top of the brickwork external infill walls and the concrete structure. **<u>These defects are characterized</u>** by bulging and local crushing of the brickwork**<u>, rotation</u>** and distortion of steel stiffeners and lintols and distortion of the aluminium window frames.
>
> (Emphasis added.)

Jones also stated at [4.4.1 of his AEIC that:

> The **bowing** of the brickwork infill walls and the vertical stiffeners leads to **rotation** or lateral movement of the horizontal stiffeners, cracking and debonding of the render finish, damage to suspended ceiling and plaster finishes internally.
>
> (Emphasis added.)

251 However, in cross-examination, Jones then tried to distance himself from the notion that was any "rotation" or "twist" at all in the distortion recorded in photographs and seen in his own inspections. Instead he insisted that the form of distortion seen was similar to the distortion seen in Dr Chiew's laboratory reports which was not rotation but a combination of local web crushing and side-sway.

252 Although Jones was reluctant to agree that there was any torsion in the horizontal steel stiffener, he still accepted that torsional moments at least were induced in the member under certain offset conditions. However, even after saying that, Jones still insisted that this was side-sway, and refused to accept that there was torsion present in the set-up.

MR MANIAM:	Mr Jones, if there is deflection, for instance, such that there is some separation of the brick wall from the stiffener, would that allow for torsional defects to develop?

A. If the centre line of the applied load is then offset from the shear centre of the section, then yes, a torsional moment will develop.

COURT: But we have seen the top plates move laterally relative to the bottom plate, so you have -- they call it failure --

A. It's what I call "side sway".

COURT: Ah, side sway. And when the side sway happens, loaded with the bricks on the top, which is now off centre -- which is now misaligned, that torsion has to be transmitted to the end somewhere.

A. If a torsion develops, yes, your Honour.

COURT: Yeah. This torsion has developed; right?

A. What actually happens is if the bricks' load centre line is moving out and the member is distorting, well, actually the shear centre is moving as well. So it's not like the member stays fixed and the load is applied outside to give the eccentricity -- (simultaneous speakers - unclear) --

COURT: -- and relieve the torsional stress?

A. It will, yes, at the same time. And my view is that it's - what we are observing is a side-sway mechanism, not a torsional mechanism.

COURT: So side sway has created the torsion before it happened, and then after it happened, it was relieved?

A. Yes, but we would never get the torsion, because it can't rotate. The top flanges are remaining --

COURT: No, no, top flange -- all right, didn't rotate, but it side-swayed.

A. It side-swayed.

COURT: Well, after it side-swayed, the continuous weight of the brick is still applying a torsional moment, isn't it, somewhere?

A. Yes, but it's -- basically it's resisted itself at that location. You get an eccentricity and an applied moment, but you've got the brick above and below, which is also providing the restraint. It's not like free torsion. If it was free torsion, it would have to go back to the support; but because you've got the bricks top and bottom constraining the torsion --

COURT: I see. Part of the brick itself is restraining it.

A. Yes, your Honour.

(Emphasis added.)

253 When confronted with the statements in his own report citing "rotation", Jones had to admit that there was in fact rotation present which he had earlier denied.

Q. Mr Jones, for completeness, can you look at paragraph 3.3 of the same document, which is your affidavit, on page 20, where you say the general findings of the condition survey are representative of defects arising from the absence of horizontal compressible or soft joints. You go on to say that these defects are characterised by bulging and local crushing of the brickwork, the **rotation** and distortion of steel stiffeners and lintels and distortion of the aluminium window frames. Would I be correct to read this together with paragraph 4.4.1, at which we've just looked, in terms of how you see the mechanism or mode of failure?

A. Two points to this. Had there been horizontal compressible joints at the top of the brickwork, I don't believe we would have been seeing any of these defects.

Q. And your second point?

A. I think, from that, you can then link the manifestation of these defects to the comments given in paragraph 4.4.1.

Q. **Mr Jones, when you use the word "rotation" in paragraphs 3.3 and 4.4.1 in relation to the steel stiffeners, to be a bit more precise, can you tell us rotation in which plane or which type? You remember, we had an initial discussion on rotation**.

A. **Yes, yes. Rotation in all directions, predominantly rotation in the horizontal plane**.

(Emphasis added)

254 In any event, Jones admitted as could be seen in his cross-examination set out below that there would be eccentricity in vertical loading due to the lateral deflection of the horizontal steel stiffeners whether from compressive load or wind load to the extent that the load centre and the

shear centre was offset.

MR MANIAM:	Mr Jones, would you agree that **lateral deflection, whether it arises from compressive load or whether it arises from wind load, would create some eccentricity in vertical loading, because your wall is no longer in plane**.
A.	**The extent to which there is a permanent offset of the brick load to the centroid of the stiffener, yes**.
Q.	**Dr Ting has suggested that the stiffeners have low torsional capacity**, particularly where there are only three sides and there is an open section?
A.	At the end connections.
Q.	At the end connections, **and would you agree with me that that, combined with eccentricity of loading, can produce some extent of twist in the stiffener?**
A.	**No, because I believe that is constrained by the brickwork, so the flanges cannot rotate, which is why we have a side sway**.
Q.	And would you agree, Mr Jones, that that **side sway itself increases the eccentricity in loading, because your stiffener is no longer a rectangular box?**
A.	**To the extent that the load centre and the shear centre is offset, yes**.

255 In practical engineering and building construction, it would in my view be rather difficult to achieve perfect alignment without any offset.

256 JTC thus rightly highlighted that this was precisely what Dr Ting opined was part of the process of the failure of the steel stiffeners when subjected to lateral load - that side-sway deformation of the horizontal steel stiffeners would result in and/or aggravate the eccentricity in vertical loading acting on the horizontal steel stiffener (as mentioned in the preceding paragraphs on side-sway deformation).

257 JTC also rightly pointed out that the torsional forces could produce distortion of the cross-section of the horizontal steel stiffener. If the horizontal steel stiffener deflected laterally and the top of brick wall essentially remained where it was, the brick wall would incline at an angle. If that angle was excessive, that would cause cracking in the plaster at the stiffener level. As such, these torsional forces could produce distortion of the horizontal steel stiffener in the sense of lateral deflection, and cause cracking of the plaster.

(II) HORIZONTAL STEEL STIFFENERS HAD LOW TORSIONAL CAPACITY

258 Dr Ting's opinion was that the horizontal steel stiffeners had low torsional capacities, and there was a propensity for them to rotate and twist.

259 What this meant was that, if the loading had no eccentricity (*i.e.* the loads were central), and the construction and installation of the steel stiffeners was perfect, then there would be no torsion in the system.

260 However, with the practice of welding on one side, and misalignment in the construction of the steel stiffeners, then torsion would be present.

MR MANIAM:	Dr Ting, if we are considering the issue of torsion, can you just tell us what would be the effect if welding was just done on one side?
A.	Okay. Where I did the torsional calculation it was based on Wyatt's design, and I already said it was very low, very weak. Now, if you tell me that you are going to weld on one side, essentially -- remember when I actually did the calculation I saying that in the perfect condition, everything perfect, by rights I shouldn't have any torsion induced because the brick wall is aligned nicely, right centre, your horizontal stiffener is placed right centre, everything so nice. By welding on one side, you have basically introduced even though your load can be central, but your reaction is now no longer central, carrying it central, so, in other words, by doing that, you have introduced torsional moment into the system already, when originally you don't have it, if you have welded both sides, and, therefore, this essentially will aggravate your situation, even before the walls start moving laterally, because just earlier discussion on lateral wall movement was to say that, if the wall move laterally my brick wall is -- in a sense there's a delta distance away from my horizontal stiffener and so on, so there's a net effect due to this turning movement of the brick wall, but now your system itself is already introducing this, so you really need to both, if you want.

261 As Wyatt had said, if there was welding on one side only, torsional restraint became fairly difficult to define, and there would not be a reliable torsional restraint.

262 Further, with the welding as constructed, there would not be much tolerance left. Once torsional stresses were added in, I would agree with the submission of counsel for JTC that it was illusory to still count on the same welding to resist those added stresses. This would have caused the steel stiffeners to rotate and twist further in a snow-balling effect, and this would have contributed to the movement of the steel stiffeners relative to their original positions, resulting in the defects observed on site.

263 Although there was some discussion on pure torsion, warping free and warping fixed, the bottom line was that torsion was present. I accepted JTC’s position that that the weak torsional capacity of the steel stiffener system and the torsion played a part in producing the defects observed. The steel stiffeners designed with open sections at the ends were much weaker than fully closed sections for torsional loads. The torsional capacities of the end open sections were so low that the likelihood of torsional buckling due to eccentricity of vertical loads was high for H1-250 and H1-300. In my view, the design of the steel stiffeners had overlooked the rotational deformations such

that any slight eccentricity would cause the steel stiffeners to twist and warp. The weak end open sections (*i.e*. weak and relatively non-rigid end conditions) would not help to produce the torsional rigidity to help resist the rotational deformations especially those found towards the centre of the steel stiffeners. Accordingly, SEC failed to comply with section 5.9 of BS 5950-5 as it did not adequately provide for the effects of torsion in the design of the steel stiffeners.

4. POOR CONSTRUCTION OF STEEL STIFFENERS AGGRAVATED THE DEFORMATION

264 Poor construction of the steel stiffeners had further aggravated the situation. The steel stiffeners were constructed poorly, in particular, in terms of the welding at the ends of the stiffener. Clearly, if the welding had been poor, and if there had been no stub, or if there had been missing plates, the steel stiffeners would have been even more likely to fail.

265 Poor welding and construction would weaken the steel stiffeners and introduce even more eccentricity into the system, which was exacerbated by the low torsional capacity of the steel stiffeners. As Wyatt said, welding allowed a transfer of torsional moment to the stub of the horizontal steel stiffeners; clearly, if there was no welding, it would be more difficult to justify the transfer of torsional moment. If the torsional moment could not be transferred, then there would be, in my view, no torsional capacity in the steel stiffener as constructed and installed.

b. How the Steel Stiffeners caused the Defects on Site: Jones' Opinion

266 Jones' opinion was that the defects as observed on site were primarily caused by the compressive loads due to the creep and shrinkage of the r.c. frame as well as the growth of the brickwork.

267 Essentially, Jones said that the defects observed on site were caused by the following factors:

(a) excessive deflection of the steel stiffeners under compressive loads;

(b) slenderness of the webs resulting in the steel stiffeners being prone to side-sway;

(c) weakness of the steel stiffeners under compression in the packing scenario.

1. STEEL STIFFENERS WOULD FAIL DUE TO COMPRESSIVE LOADS

268 On Jones' account, the steel stiffeners would fail on account of the stresses induced in the brick wall as a result of the creep and shrinkage of the r.c. frame in addition to the growth of the brick wall. These stresses were broadly described as compressive stresses due to movement.

269 Jones said that the total difference in movements of brickwork and concrete after 6 years for a brick wall of 3.05 m height (as calculated by him) would be about 2.7 mm. This consisted of the following:

(a) shrinkage and creep of concrete (after 6 years) = 0.79 mm;

(b) moisture movement of brickwork = 1.53 mm; and

(c) thermal movement of brickwork = 0.38 mm.

270 It would be pertinent to observe that the shrinkage and creep of concrete (*i.e*. the effects of the main building structure) comprised 0.79 mm of the 2.7 mm, which would be approximately 29% of the total difference in the movements, whereas the predominant effect would be due to the moisture and thermal movement of the brickwork itself, which would account for 1.91 mm out of the 2.7 mm, which would represent approximately 71% of the total difference in movements. Relatively speaking, the predominant effect (71%) of all the movement originated, according to Jones, from the brickwork itself and only a small percentage of 29% of the movement could be attributed to the shrinkage and creep of the building structure. I further noted that Jones did not appear to need a lot of information on the building before he could calculate these movements and I would therefore also not expect the designer of the steel stiffeners to encounter any difficulty in calculating such movements, if he had wanted to take them into account in his design. Indeed, if the steel stiffener designer, Wyatt, needed any further (interface) information from JTC to enable him to properly design the steel stiffeners, he should have asked JTC to supply him whatever additional information he needed.

271 Counsel for SEC rightly pointed out at paragraphs 290 and 291 of its reply submissions that the above figure of 2.7 mm movement in total as computed by Jones was in fact based on the worst case scenario because the thermal expansion of the brick wall could in fact be disregarded as the r.c. structural frame would expand more than the bricks therefore leaving a gap in between. Apparently, according to SEC, even by taking account 50% of the thermal effects, the design of the steel stiffeners could still accommodate the effects of the brick work movement as shown in their calculations at paragraph 295of the reply submissions as follows:

(a) H1-300 at SS2000 external wall:

$$[1.53 + (50\% \times 0.38)] \times \frac{5.35}{3.05} = 3.0\text{mm}$$

3.0mm – 1mm = **2.0mm** < 3.7mm (vertical capacity)

The spare capacity available appears to be 1.7 mm after allowing 1 mm for the soft mortar joint.

(b) H1-150 at staircase B:

$$[1.53 + (50\% \times 0.38)] \times \frac{5.82}{3.05} = 3.3\text{mm}$$

3.3mm – 1mm = **2.3mm** < 5.0mm (vertical capacity)

The spare capacity available appears to be 2.7 mm after allowing 1 mm for the soft mortar joint.

(c) H1-250 at external toilet wall:

$$[1.53 + (50\% \times 0.38)] \times \frac{5.20}{3.05} = 2.9\text{mm}$$

2.9mm – 1mm = **1.9mm** < 2.0mm (vertical capacity)

The spare capacity available appears to be 0.1 mm after allowing 1 mm for the soft mortar joint.

(d) H1-300 at internal toilet wall:

$$[1.53 + (50\% \times 0.38)] \times \frac{5.82}{3.05} = 3.3\text{mm}$$

3.3mm – 1mm = **2.3mm** < 5.0mm (vertical capacity)

The spare capacity available appears to be 2.7 mm after allowing 1 mm for the soft mortar joint.

272 On this submission, it would also appear that the shrinkage and creep of the building structure could also be accommodated (except for the H1-250). The shrinkage and creep for a 3.05 wall height was 0.79 mm according to Jones. Hence, the shrinkage and creep for walls of different heights on a proportional basis would be as follows:

(a) For a wall height of 5.35 m, shrinkage and creep of the building structure would be 1.38 mm. This could be accommodated within the spare capacity of 1.7 mm for the H1-300 at SS2000 external wall.

(b) For a wall height of 5.82 m, the shrinkage and creep of the building structure would be 1.5 mm. This again could be accommodated within the spare capacity of 2.7 mm for the H1-150 at staircase B and the H1-300 at internal toilet wall.

(c) For a wall height of 5.2 m, the shrinkage and creep of the building structure would be 1.34 mm. This could not be accommodated within the spare capacity of 0.1 mm for the H1-250 at the external toilet wall.

273 From SEC's own submission, it would appear that its own case, attributing the cause of the failure entirely to the shrinkage and creep of the building structure and moisture movement of the brickwork with 50% thermal movement from the brickwork (which actually could well be disregarded), was not well founded as there was capacity in the steel stiffeners to account for all these movements, with the exception of the H1-250 at the external toilet wall.

274 Jones said that the net movement due to the creep and shrinkage of the concrete structure and the expansion of the bricks would be 2.7 mm for a 3.05 m height wall, with some consideration for the elasticity of the mortar and air gaps in the mortar.

COURT: I noted your 2.7 mm is actually net, so as part of the net, the earlier components would have included the compressibility of the brickwork, then the expansion due to thermal and due to moisture; right? The only thing is the assumed crushing is not there, because you don't know the amount of crushing and therefore that is not taken into account, plus any air gaps, if there are any air gaps to begin with.

A. Well, the modulus of elasticity of --

COURT: Air gap mean the incomplete plastering; that is the difference of some air gaps here and there.

A. Yes, your Honour.

COURT: So these two effects are not there.

A. It's partially there because the modulus of elasticity of brickwork **takes some account of that**; it assumes that you don't get perfect construction in brickwork.

COURT: The compressibility which you used in the brickwork plus the mortar takes account of any air gaps that might be there due to workmanship, as opposed to the air gaps within the plaster itself? The plaster has it own air gap but when you say the elasticity, you included the air gaps within the plaster, plus the incomplete plastic, if any at all?

A. Yes. Your Honour, these E values which are used to evaluate compression under elastic loads, they are all derived from tests. So it is actually tests on actual mortar which will conclude some elasticity. The actual amount of air gaps, well, that is an unknown, but **there is some consideration there**.

(Emphasis added)

275 In this regard, Jones calculated that the expansion of the brickwork and the shrinkage and creep would result in a load of 6.2 N/mm^2, some 65 to 70 times more than the dead weight due to the self-weight of the brickwork.

276 Jones then calculated that for a combined vertical movement of 2.7 mm, the lateral bow of the brick wall would be 56 mm; further, due to the crushing / cracking of the brick / mortar bedding joint, the 56 mm would be reduced by about half.

277 JTC's position was that the 2.7 mm vertical movement would not have occurred. On the other hand, SEC's position was that the 2.7 mm vertical movement would have been accommodated if JTC had expressly provided for movement joints in the original design (with r.c. stiffeners), or if JTC had not rejected movement capability of either the steel stiffeners or where the brick wall and stiffeners abutted.

(I) MOVEMENT CAPABILITY REJECTED BY JTC

278 SEC appeared to be saying that the original Dyntek proposal would have allowed movement capability (*i.e.* which would have allowed the vertical movement of 2.7 mm without problems), but JTC rejected this. Therefore, if the defects were caused by the compressive loads in the set-up, SEC could not be held liable to JTC for the defects.

279 In this regard, SEC's argument seemed to rest on the following propositions:

(a) Welding of the stub to the steel stiffener was required by JTC (where in the original steel stiffener design, the connection of the stub to the steel stiffener was that of a movable "sliding sleeve connection"), such that a "tight fit" could be achieved for the vertical steel stiffener between the r.c. structure at each floor level; and

(b) This was to assuage JTC's concerns about leakage problems if the stub and steel stiffener were not welded together.

280 Jones provided an explanation of this at paragraph 4.4.4 of his 1st AEIC; that:

> The steel vertical stiffeners, **as they are welded to achieve a tight fit between the reinforced concrete structure at each floor level** [ref. Dyntek / W.P. Brown Singapore drawing "Typical Connection Details (100mm walls), revision F dated 14 April 1999] **will also bow as the reinforced concrete columns shrink and creep between each floor level**. However, the contract documents make no reference to the need for horizontal compressible joints. Correspondence between Sembcorp and JTC indicate that this **"tight fit" was instructed by JTC to mitigate water ingress through the wall from the building exterior**.
>
> (Emphasis added)

Also, at paragraph 10.7 of Jones' 1st AEIC:

> The original Dyntek stiffener proposals submitted to JTC by SembCorp included a movement facility at the top of the vertical stiffener to permit relative vertical movement between the structure and the brickwork. **JTC rejected this proposal on the grounds of potential leakage**…

(Emphasis added)

281 On the evidence, I accepted JTC's position that it had not rejected any movement capability (which SEC said was afforded by the original steel stiffener design).

282 I noted that Jones conceded that, in his view, the correspondence between WP Brown and Dyntek in August 1998 where they were working out what the connectors would be in a load-bearing scenario, showed that "*they were attempting to adopt the principles from the standard details and then beef them up to carry the loads they had to carry*". As such, it was actually Jones' view that, by August 1998, the designer was already not contemplating the use of the standard sliding sleeve connection (as *per* the Dyntek catalogue) for the Development. Thus, Tong's comment of "*not acceptable*" for "*leaking problems*" could not have been an attempt to reject the movement capability afforded by the standard sliding sleeve connection in the standard Dyntek catalogue. The point was really simply just that at the time of the submissions for approval, WP Brown was already not looking at the standard Dyntek detail, but rather, was considering the non-standard sleeve connections at the stubs (which required calculations).

283 In any event, the steel stiffeners would not have been waterproof since it was undisputed that the webs and flanges of the steel stiffeners were to be welded together by intermittent welding (rather than continuous welding), and would not have been impervious to water ingress (if that was indeed JTC's concern at all). This would have made it pointless for the top of the vertical steel stiffener (of which the webs and flanges were also not welded together by continuous welding) to have been welded to the stub if the objective was to prevent water ingress. Further, the removal of the bottom plate at the ends of the horizontal stiffeners, as well as the slit at the bottom of the stubs and non-continuous welding, would mean that the steel stiffeners as designed and installed were not waterproof at all to begin with.

284 In any event, the designer of the steel stiffener system, Wyatt, did not think that there were any instances where the compression of the vertical steel stiffeners were causing any significant problems, and did not recall seeing instances where vertical steel stiffeners under compression gave problems.

(II) WIND LOAD WAS SECONDARY FOR BEING A TRANSIENT LOAD

285 In this regard, Jones considered that wind load (which was a transient load, not permanently applied) was merely of secondary impact to the effects of the "packing loads" arising from shrinkage and creep of the primary structure and expansion of the bricks within the infill walls. Jones did not believe that wind load would cause any permanent defects to the brick walls.

286 However, I noted that Jones was clearly inconsistent on this point, as he admitted subsequently that once the steel stiffeners were pushed out as a result of wind load, it would be very difficult to pull the stiffeners back.

2. STEEL STIFFENERS SLENDER AND PRONE TO SIDE-SWAY

287 Jones accepted that the steel stiffeners were too slender and were prone to side-sway.

3. STEEL STIFFENERS WEAK UNDER COMPRESSION

288 Taking one wall panel actually found on site, Jones himself calculated that the applied stress due to the shrinkage and creep of the concrete structure acting in tandem with the expansion of the brick wall (of height 3.05 m) above the horizontal steel stiffener, was 6.2 N/mm^2. This applied stress of 6.2 N/mm^2 would generate a load equivalent to 620 kN (6.2 N/mm^2 x 1000 mm (per m run) x 100 mm (width)).

289 Similarly (as the applied stress varied linearly with the height of the brick wall), the applied stress due to the shrinkage and creep of the concrete structure and the expansion of the brick wall (of height 2.2 m) below the horizontal steel stiffener, was 4.5 N/mm^2. This applied stress of 4.5 N/mm^2 would therefore generate a load equivalent to 450 kN.

290 On the other hand, the horizontal steel stiffener (H1-300) in that wall dimension was tested by Dr Chiew to have a maximum capacity of between 63.3 kN and 65.1 kN only.

291 Wyatt said however that the steel stiffeners would have some vertical displacement capacity (see D-23) to take the movement in brick wall calculated by Jones.

292 This meant that (in that location), the applied stress calculated by Jones was some 6 to 8 times greater than the capacity of the horizontal steel stiffener.

293 Consequently Jones had to admit that the steel stiffeners were weak under compression (in the packing case) and were in fact the weakest point in the system, as compared to the brickwork. The steel stiffeners would suffer from side-sway; and according to Jones, would exhibit both vertical and lateral displacement of the flanges relative to each other.

A.	*The first thing, your Honour, is the calculation of what happens to the horizontal stiffener embedded in the wall when you have brickwork both above and below, you have the packing case and the effect that would have on the bow of the wall,* ***bearing in mind you would get side sway and vertical compression of the stiffener****. The actual -- from the panel above the stiffener, the actual load generated by the 2.7 mm compression would be 620 kilonewtons. For the panel below the stiffener, the load generated would be 450 kilonewtons from the expansion of that panel as well as the shrinkage and creep acting from below effectively pushing up.* ***The steel horizontal stiffener only has a capacity, based on Dr Chiew's tests, of 64 kilonewtons, so what I think would happen is that as you approach 64 kilonewtons****, you would get relaxation of the shrinkage, creep and expansion loads, such that bow does not occur, and* ***you'd get side sway and vertical movement of the steel stiffener****, which is what you are actually seeing on site.* ***So the horizontal steel stiffener is compressing, swaying sideways, before the big creep and shrinkage load and the expansion of brickwork load can be generated, so it relaxes. So the weakest point in the system is the steel stiffener…***
COURT:	*Now we are looking at, in very real terms, which is the weakest link.. After all this, the conclusion is?*
A.	*Well, the conclusion is -- my conclusion is still the same, that the distortion of the horizontal stiffeners is caused by this load.*
COURT:	*By the creep?*
A.	*Shrinkage and creep and the brick expansion.*
COURT:	*Which couldn't reach it but forced it out?*
A.	*Well, it's actually the load imparted --*
COURT:	*Forced the lateral side sway?*
A.	*Yes. The load imparted on the steel stiffener is sufficient to cause that side sway.*
COURT:	*There is no need for a bow; the bow doesn't even reach?*
A.	*The bow doesn't even come into play with the steel --*
COURT:	*So the wall remains straight, it is just pushed out as it tries to expand?*
A.	*Yes, your Honour…*

(Emphasis added)

294 Accordingly, if one were to consider the use of the various materials present in a brick wall in a packing situation (consistent with the situation as found on site) from the perspective of compressive strength of each material, the steel stiffeners once again fell short of the mark.

295 Dr Ting had calculated at P-23, Table 1 ("*Compressive Strength When Used as Packing Member*"), that the compressive strength of the steel stiffeners was significantly less than that of r.c. (1,584 kN/m) and brickwork (150 kN/m) for the H1-250 (32 kN/m) and H1-300 (43 kN/m) horizontal steel stiffeners, and only comparable to brickwork for the H1-150 horizontal steel stiffeners (149 kN/m). Further, Dr Ting also clarified in re-examination that the compressive strength of even mortar alone was 415 kN/m.

296 What the above in effect meant was that the steel stiffeners (at least for the H1-250 and H1-300 horizontal steel stiffeners), having the lowest compressive strength in the whole set-up (in a packing case), would be the first to fail - even before the brickwork failed.

297 On a comparison of the various materials present in the packing scenario then, it was clear that the horizontal steel stiffeners were indeed the **weakest link** in the whole system, as compared to the brickwork and mortar, not to mention r.c. It was plain to me that the originally intended r.c. stiffener, if not substituted with the steel stiffeners, would have much superior compressive strength when compared with the steel stiffeners. In the packing scenario and under so called packing loads, the solid r.c. stiffeners would not have failed under compression or by side-sway unlike the steel stiffeners. I would not believe that the solid r.c. stiffeners would ever have distorted from a rectangular cross-sectional shape into a parallelogram shape under the packing or compression loads found on site.

298 Jones himself had also carried out similar calculations in respect of the compressive strengths of the different elements within the brick wall panel matrix.

299 In Jones' latest set of calculations, at "*Table 5: Ultimate Compressive Strength of Packing Material*", Jones provided the calculated compressive strengths of r.c. (1,050 kN/m), H1-150 (101 kN/m), H1-250 (46 kN/m) and H1-300 (61 kN/m) horizontal steel stiffeners, as well as brickwork (52.6 kN/m). Jones also commented next to the table that "*Brickwork and steel stiffeners therefore have compatible stiffness*".

300 Counsel for JTC however submitted that its requirements were never that the steel stiffeners were to have "*compatible stiffness*" with the brickwork. What JTC required of SEC was a stiffener system that was suitable for the walls in question, and moreover would function even if the bottom brick wall panel under the stiffener was removed. Clearly, SEC did not meet that standard, and JTC should be entitled to damages in respect of the same.

301 Despite it being undeniable that the steel stiffeners were indeed the "weakest link" in the system, SEC still tried to salvage the situation, but this was to no avail. In fact, it was suggested to Jones that when he described the steel stiffeners as the "weakest link", what he really meant was that it was the "*first point of contact to alleviate compressive stresses*", and there was no deficiency or inadequacy ascribed to the same. Jones of course agreed, and equivocated that all that he was saying, by using the term "weakest link", was that the most compressible element would give way first, to relieve the stress.

MR SOH: Mr Jones, when you described the brick or the stiffeners as the weakest link, are you using this term in the context of it **being able to be the first point of contact to alleviate compressive stresses**?

A. **Yes, I am.**

Q. Is it your intent, when using the term "weakest link", to ascribe some sense of deficiency or inadequacy about the brickwork or the stiffeners, when you --

A. **No, all I am saying is that the most compressible element will give first, to relieve the stress.**

MR SOH: … over the last two days, the term "weakest point" or "weakest link" has been used. So I'm just confirming with Mr Jones he is using it in the context of compressibility.

(Emphasis added)

302 However Jones or SEC sought to characterise the point, it remained in my view an admission that the steel stiffeners were not fit for their purpose. There was no justification for the substitution of steel for r.c. if the stiffeners provided were no better than brickwork, resulting in the stiffeners being the "weakest link". The use of steel stiffeners simply led to much higher repair costs to JTC, as compared to merely replacing the top course of bricks and re-plastering (if that was even required) in the case of r.c. stiffeners.

c. Steel Stiffeners would have failed with vertical movement of 2.7 mm

303 Although Jones' theory was that, due to creep and shrinkage of the r.c. structure and expansion of the bricks, there would be an overall vertical movement of 2.7 mm in respect of a brick wall panel of 3.05 m height. In truth, the vertical steel stiffener would have failed already with a movement of 1.7 mm.

304 If there was no change in the vertical loading condition, Jones would not expect any additional vertical displacement of the horizontal stiffener to occur after the brickwork and plastering was completed.

305 Thus, Jones considered that the effect of shrinkage movement and brick expansion were 4 times that of wind. Jones, however, had not performed the calculations for wind, as he said that it would have taken him several hours. He approximated the value to 1, stating that if he had had time to do a detailed analysis, the value was likely to have gone below 1.

306 If Jones were to be right and the defects observed on site were due to the compressive stresses in the brick wall, there would be movement of a magnitude of 1.7 mm due to the creep and shrinkage of the r.c. frame and the expansion of the vertical steel stiffeners (manifesting as axial load), which would cause the failure of the vertical steel stiffeners in any event.

307 On either JTC's expert's account or SEC's expert's account, the vertical steel stiffener would

have failed.

308 Hence, despite the difference of expert opinion between Jones and Dr Ting on whether it was the compressive load or lateral load which was the primary cause of the failure of the steel stiffeners, the lowest common denominator (as it were) was that the steel stiffeners were not fit for their intended purpose, and SEC would be liable to JTC for the same.

1 VERTICAL STEEL STIFFENERS WOULD HAVE FAILED WITH VERTICAL MOVEMENT OF 1.7 MM

309 Keeping in mind that the vertical steel stiffeners were meant to be and were in fact installed between r.c. members (such as floor slabs and tie beams), they should have been designed to be able to withstand expected movements of the r.c. structure.

310 However, even leaving aside any damage to the finishes or brick walls, on Jones' creep and shrinkage theory, the vertical steel stiffeners would themselves have failed for excessive deflection already (again, leaving aside the vertical movement of 2.7 mm due to creep and shrinkage of the r.c. frame and expansion of bricks, as *per* the discussion in the preceding passages).

311 In essence, this was because the thermal expansion for steel is 2.4 times that for brickwork, for the same temperature variance. This would arise from the different coefficients for thermal expansion for steel (of 12×10^{-6} /°C - from section 3.3.3 of BS5950-5, which Jones had also used) as compared to the 5×10^{-6}/°C for brickwork (used by Jones).

312 Jones had alluded to that in his 1st AEIC, at paragraph 4.3.2 (page 27), albeit less clearly as follows:

> The steelwork vertical stiffeners attract greater load than the reinforced concrete stiffeners due to the alleviating, or strain relaxation effect of the shrinkage and creep of the vertical concrete stiffener.

313 Jones himself also admitted as much during cross-examination.

Q. Yes. We are talking, just to be clear, about the compressive forces which you have calculated arising from net expansion [of bricks]?

A. Combined with shrinkage and creep of the concrete.

Q. On that point, Mr Jones, if I can take you back to your affidavit and your first report, looking at page 27, taking the numbering at the top right-hand side, I'm looking at what you will see as subparagraph 2, this is actually 4.3.2.

What you see there is that the steelwork vertical stiffeners attract greater load than the reinforced concrete stiffeners due to the alleviating or strained relaxation effect of the shrinkage and creep of the vertical concrete stiffener.

What I understand from that, Mr Jones, is that the **<u>vertical concrete stiffeners would shrink or creep in the same direction as the vertical concrete columns?</u>**

A. **<u>That is correct but to a lesser extent</u>**.

Q. **<u>What would the behaviour be of the vertical steel stiffeners?</u>**

A. **<u>They do not shrink and creep</u>**.

Q. **<u>Would they in fact expand on account of temperature effects?</u>**

A. **<u>Expand and contract</u>**.

Q. And, so, as a threshold position **<u>the steel stiffeners come under greater stress than the reinforced concrete stiffeners, these are the verticals, if we were just looking at shrinkage and creep?</u>**

A. **<u>Correct</u>**.

(Emphasis added)

314 This was then fleshed out quantitatively. According to Jones, based on the compressive stresses for the wall panel as referred to at page 335 of his 1^{st} AEIC, the vertical steel stiffener of height 5.6 m would suffer a lateral deflection of 60 mm. The deflection ratio would be H/90 or thereabouts. This was clearly unsatisfactory; Jones himself conceded that deflections of this magnitude might create a problem. This was not surprising, since by Jones' standards of deflection ratios, some damage to the finishes / plaster would already be occurring at around half the lateral deflection suffered (*i.e*. with a deflection ratio of around H/180).

A. If we can look at page 319 of my first affidavit, which is the calculation for the brickwork with 3.05 metres high, if we take those parameters and look at the vertical stiffener, we can assume that the shrinkage and creep of the concrete is the same value at 0.79 and the moisture movement of the brickwork has no relevance, so that goes; the thermal movement of brickwork would increase because of the relative coefficients of thermal expansion, and with the help of Mr Maniam and Dr Ting in the break, we found in BS5950 that the coefficient is 12 times 10 to the minus 6 per degrees C for the steel, so the 0.38 will increase by the ratio -- will increase to 0.9, the 1.53 won't occur, so the **<u>net total is then 1.7 mm</u>**.

COURT: Can you work out the deflection for the 5.6 metre vertical stiffener length? I suppose you must ratio the vertical shortening of 1.7 over 5.6 now and calculate the bow for 5.6. You have to go back to your exhibits this morning. That will be D --

A. D43, case 2A, which for case 2A would be 60 mm, and for case 2B --

COURT: So, **<u>for steel it would be 60 mm?</u>**

A.	**<u>Yes</u>**, your Honour. And for case 2B it would be 35 mm.
COURT:	And you do the H check, the serviceability check.
A.	(Pause.) It seems to be around the **<u>H over 90</u>**.
COURT:	L over 90?
A.	90
COURT:	**<u>That would be below your 150</u>** --
A.	**<u>Yes</u>**.
COURT:	-- **<u>which may create a problem</u>**.
A.	**<u>Yes</u>**.

(Emphasis added)

2. HORIZONTAL STEEL STIFFENERS WOULD HAVE FAILED WITH VERTICAL MOVEMENT OF 2.7 MM

315 With vertical movement of 2.7 mm due to creep and shrinkage of the r.c. frame and the expansion of the bricks, the defects observed would necessarily have been even worse.

316 When considering the effect of 2.7 mm vertical movement on the brick walls, it was apparent from the outset that the weakest link in the on-site situation was the steel stiffener. With vertical movement of 2.7 mm, Jones admitted that the horizontal steel stiffeners would side-sway first.

317 Jones tried to salvage this "weakest link" admission by relying on the lack of compressible joints as the reason behind the defects observed.

318 Jones' position was indefensible. Firstly, the Code did not make the provision of movement joints mandatory, but rather it was something which the designer ought to have considered; secondly, both Jones and Wyatt admitted that movements joints were not the industry practice in Singapore; thirdly, movement joints as suggested by SEC (at the underside of the structure where the brick walls abut) would not have relieved the stress to the vertical steel stiffeners.

(I) QUESTION OF WHETHER STEEL STIFFENERS WERE FIT FOR PURPOSE

319 If Jones was indeed right and the defects caused could have been entirely prevented by the provision of movement joints, given that Wyatt was aware that an r.c. frame was to be used for the Development (and further, of the dimensions of the r.c. frame), and that Wyatt was aware that the provision of movement joints was not industry practice in Singapore (and he did not expect such joints in the walls he was designing the stiffeners for), Jones must in effect have been saying that Wyatt was negligent (or at fault in some way) in expecting his design to function properly without movement joints. In my judgment, it could not be JTC's responsibility to re-design the Development to

accommodate Wyatt's steel stiffeners, which were proposed by SEC and were to be designed and constructed by SEC.

320 If there was a vertical downward displacement of the r.c. beam, it would appear that even adding a compressible joint as suggested by Jones would not help the problem of the vertical displacement on the vertical steel stiffener, which was rigidly fixed to the r.c. beam. Jones tried to deflect attention away from the steel stiffeners by saying that if the vertical steel stiffener was built with a sliding sleeve connection to the r.c. beam at the top, or if the vertical steel stiffener had a compression joint at its top, or if the brick walls were thicker to begin with, then "it would work". However, it did not seem to me that these sliding sleeve connections to the r.c. beam at the top of the vertical stiffener were contemplated to begin with, and if the steel stiffeners had to have all these sliding sleeve connections to work, then they should not have been designed and constructed as they were. Jones responded that if had r.c. stiffeners had been used instead, the end result would have been bulging walls; with steel stiffeners, we instead ended up with *"distorting stiffeners for steel stiffeners, which is the weakest link"*.

Q.	Mr Jones, it's JTC's position in these proceedings that defective design and construction of the steel stiffeners led to the defects that are observed. I take it you would not agree with that?
A.	I disagree because my view is that had there been horizontal compressible joints at the top of the brickwork, these defects would not have manifested themselves.
Q.	Mr Jones, there is one --
COURT:	-- want that a bit more detailed. Your compressible joints may be effective at the centre of the beam, I'm not sure whether it's going to be effective nearer the ends, you know, because you have the fixed pin joints there, it's not going to move downwards where the pillars are. Similarly, close to your vertical pillar, I'm not sure whether or not you've got that much of a possibility, even adding compressible joint depends on your amount of movement you're talking about, if it's 2.7, you're going to have the same problem at the centre of the beam, at the vertical stiffener still.
A.	Well, that's the issue with the Dyntek detail for the sliding design, such that the top of the stiffener can compress.
COURT:	But this one is not (unclear) that way.
A.	I know it's not. That's what I am saying, your Honour.
COURT:	Whose problem is this?
A.	That's not really a technical issue.
COURT:	Oh, all right, fine. So we are stuck with the problem of having a fixed vertical stiffener there?

A. Yes, your Honour.

COURT: You may have your compressible joint though, but the compressible joint only acts near the centre, where the beam can deflect, elsewhere it cannot - same problem?

A. But when I'm saying a horizontal compressible joint, I'm saying that the vertical stiffener as well will need that.

COURT: Yeah, but if you don't have that -- I mean I'm looking at the real (unclear) as design. So I've got a problem mentally, right?

A. Yes, your Honour --

COURT: You may say earlier on you tell me your work, I look at it carefully, not necessary, correct?

A. It would only work if you had a much thicker wall.

COURT: A much thicker wall?

A. If you had a much thicker wall, you can design that wall to carry this load.

COURT: But we know as we specified, 115 mm.

A. Yes, your Honour, yes.

COURT: And then, whoever designed it would design it, so I am saying that, looking at **the compressible joint is not going to work well either, correct?**

A. **It wouldn't work unless you have also the vertical stiffener with the sleeve at the top**.

COURT: Yeah, but I don't have it -- the point I don't have it.

A. But we don't have the movement joint anyway -- (unclear -- simultaneous speakers) --

COURT: Yeah.

A. -- the brickwork.

COURT: **Even you have at the centre, I don't have it the brick wall, so it can't solve it, so the answer is don't use it. Then the metal stiffeners shouldn't be used unless you have all these things in, otherwise you don't use it. It's not going to work, that's what I mean, that's where I'm coming from, it's not going to work**.

A.	Then we have the problem that if we have a alleviation because of the compression and side sway of the horizontal steel stiffeners, but if we had concrete stiffeners which are relatively incompressible, then we're -- (unclear -- simultaneous speakers) --
COURT:	Then the walls would be the one that's bulging out?
A.	-- going to get bulging walls, so we can't win.
COURT:	Okay, okay. **<u>So we get bulging walls for concrete stiffeners?</u>**
A.	**<u>And distorting stiffeners for steel stiffeners, which is the weakest link</u>**.

(Emphasis added)

321 JTC rightly submitted that, upon an enquiry as to whether the steel stiffeners were fit for their intended purpose, the only relevant question in the enquiry was whether or not, given that the factual matrix was known to the designer (*i.e.* no sliding sleeve connection between the vertical steel stiffener and the r.c. beam, no movement joints, 115 mm thick brick walls, etc.), the steel stiffeners were fit for their intended purpose if they ended up distorting and being the weakest link in the matrix.

322 I fully agreed with JTC's submission that the answer to the question of whether the steel stiffener system was fit for its purpose must be in the negative.

3. SOME ACCOMMODATION OF THE VERTICAL MOVEMENT OF 2.7 MM

323 Apart from the analysis of whether or not the vertical movement of 2.7 mm would have caused the horizontal steel stiffeners to fail and/or the brick walls to be damaged, I agreed with the submission of counsel for JTC that it was not likely that there would have been the full vertical movement of 2.7 mm in any event due to the relief provided by:

(a) Crushing of the mortar (soft joints) between the bricks;

(b) Air voids in the mortar between the bricks;

(c) Vertical and/or lateral deflection of the horizontal steel stiffeners themselves;

(d) Vertically-placed or flipped bricks;

(e) Lateral deflection of the brick walls.

These reasons are set out in greater detail as follows.

324 Soft joints in the form of mortar between the bricks would be crushed under compressive load, and this would provide tolerance for movement. Jones himself acknowledged this.

Q. Now, what would the mechanism be like as those loads build up, if we have the compressive strength figures for the brick wall, i.e. the mortar plus the bricks, on the one hand, and the H1-250 steel stiffener as calculated by you?

It wouldn't suddenly appear even at the limit of the steel, so it's a progressive thing, can you help us just work through how that would play out in the wall?

A. Well, the load would be gradually increasing from day 1 onwards. **The load gradually builds up until some relief is found, and relief is found by, in the first instance the weakest link in all this is actually the mortar in the brickwork, so you find that the mortar in the brickwork starts to deform earlier**.

Q. What happens then?

A. The load continues to build up until such time as the load is sufficient to start deformation of the frame around the brickwork, and with that period you may get bowing of the brickwork as well, so you've got a **system of loads increasing, loads relaxing, loads increasing loads relaxing through time, until such time as the load in one particular element is over its ultimate capacity**.

…

Q. And so, again, **the weakest layer, from your perspective would be the mortar, so that there would be some give there**?

A. **Yes**.

Q. **And as the mortar gives way it relaxes or relieves the stress to some extent, and the stress builds up again thereafter**?

A. **Yes**…

(Emphasis added)

325 In this regard, Jones said that the mortar joint at the top of the brick wall (underneath the r.c. beam or concrete structure) was generally the weakest as the mortar would have to be forced into the gap. The mortar joint thus behaved like a compressible joint. SEC's own expert acknowledged that the mortar joint at the top of the brick wall would have some tolerance for deflection.

COURT: You cannot assume 2.7 is at one point. It's spread out over the whole wall.

A. Logic dictates that it will be spread out, but the **most difficult mortar joint to construct is one at the top and that's generally the weakest**. Normally, when you are buttering the wall with a trowel and mortar you can --

COURT:	Yes, you can do it. The most difficult is to splash the mortar --
A.	At the top you have got to force it in. That's the most difficult spot to get a good mortar joint.
COURT:	Is if anything else fails, that would be the part that would fail.
A.	That's normally the one that would fail first, your Honour.

(Emphasis added)

326 Voids in mortar (air gaps) would allow further tolerance for movement. Dr Ting said that under compression, the air voids in the mortar would be squeezed out.

327 Jones also said that, at a vertical deflection of 4.65 mm due to creep and shrinkage and dead weight of bricks, the forces would add up to about 60 kN, which was within the compressible range of the mortar and still a far cry from the 180 to 360 kN maximum compression for mortar to crush entirely. However, that would be assuming perfect mortar; because of workmanship, there would inevitably be a few voids here and there, and the maximum compression at which the imperfect mortar would crush was at about 60 kN.

328 That would give about 4 mm lateral deflection (purely as result of vertical load, without wind load).

329 Jones acknowledged that when you get side-sway of the horizontal steel stiffeners, you would also get movement relaxation, and the deflection would be less than calculated. Jones even conceded that "*that mechanism of side sway actually then relieves the load that's applied as well*".

330 Vertically-placed/flipped bricks provided further tolerance for movement. Thus, Jones commented that with flipped bricks standing on their ends, a soft joint was achieved; although he could not say whether the bricklayers intended that effect, but it would fill the gap and it would work. So, from the perspective of accommodating movement, it would still function better than a brick laid horizontally, because the bricks on end were weaker in terms of compressive strength.

331 Subsequently, Jones tried to say that brickwork was failing before the mortar itself, and that this was exactly the sort of evidence that one would identify as being due to shrinkage and creep of the concrete structure and expansion of the brickwork. However, when questioned further by the Court, Jones agreed that the crushing of the bricks at the top of the wall at the staircase as observed was due to the bricks on top being put on end (with the holes in the brick being in the horizontal rather than the conventional vertical direction). This meant that when the bricks in the top row were put upright, they were actually weaker than the mortar under compression.

332 Lastly, when the brick wall deflected laterally, the bowing would also relieve the stresses to an extent.

333 All these factors would relieve the stresses in the brick wall to some extent, such that the full 2.7 mm vertical movement due to creep and shrinkage of the r.c. frame and the growth of the brickwork would not even occur, although I did not discount that some residual vertical movement might remain where full relief was not possible from the mechanisms described above.

(I) CAPACITY OF BRICK WALL WAS INSUFFICIENT

(A). AXIAL STRESS LIMIT WAS REDUCED BY JONES

334 Jones said that the capacity of the brick wall was insufficient to withstand the vertical load imposed on the brick wall, because the brick wall was too slender. In other words, according to Jones, the allowable stress which the brickwork could take (*i.e.* the axial stress limit) was so low such that the capacity of the brick wall ended up being insufficient. As such, Jones calculated (at page 92 of D-29) that the axial stress limit of the brick wall of 3.05 m was 0.457 N/mm^2. However, in his 1st AEIC filed in May 2007, Jones had calculated the axial stress limit to be 1.3 N/mm^2.

335 This decrease of almost 3 times in terms of the axial stress limit (*i.e.* allowable stress) of the brick wall, as calculated by Jones, was due to a reduction in magnitude of various factors which Jones had used in his calculations. Accordingly, the final result he arrived at was reduced drastically.

336 In his 1st AEIC filed in May 2007, Jones calculated the slenderness ratio of the wall to be 30.5, and thus obtained the corresponding capacity reduction factor, β, of 0.4 by referring to Table 7 of BS5628-1. Substituting that capacity reduction factor into the equation, he calculated the axial stress limit for the brick wall of 3.05 m height to be 1.3 N/mm^2.

337 However, by the time Jones prepared D-29, he had increased the slenderness ratio of the wall to 31, and decreased the capacity reduction factor, β, to 0.161 instead.

338 In this regard, instead of obtaining the capacity reduction factor from Table 7 of BS5628-1, Jones had instead plotted his own graph (page 92 of D-29) such that for a slenderness ratio of 31, the graph plotted by Jones reflected a capacity reduction factor of 0.161.

339 As a result, the compressive strength of the brickwork as calculated by Jones in D-29 dropped to around 52.6 kN/m which was a corresponding reduction of almost 3 times. This was the basis for Jones' statement at Table 5 that "*Brickwork and steel stiffeners therefore have compatible stiffness*", since the compressive strength of the H1-250 and H1-300 horizontal steel stiffeners were calculated by Jones to be 46 kN/m and 61 kN/m respectively.

340 However, if the compressive strength of the brickwork was calculated on the basis of the 1.3 N/mm^2 axial stress limit found in Jones' 1st AEIC, the compressive strength of the brickwork would be some 3 times that of the H1-250 and H1-300 horizontal steel stiffeners. This would in effect mean that the horizontal steel stiffeners would not have "compatible stiffness" with the brickwork, but would instead fail before the brickwork.

341 It was telling that Jones had re-done his earlier calculations for the axial stress limit of the brickwork in his 1st AEIC such that with D-29, he could contend that the new compressive strengths of the brickwork and steel stiffeners showed the brickwork and steel stiffeners to have "compatible stiffness".

(B). BRICKWORK HAD GREATER CAPACITY THAN THAT CALCULATED BY JONES

342 JTC submitted that the brick walls in fact had greater capacity than what Jones gave them credit for. According to Jones, the capacity of the brick walls was apparently insufficient based on his calculations. However, counsel for JTC emphasised that Jones' calculations were based on extreme assumptions, including that:

(a) The permissible slenderness ratio of the brick walls was exceeded; and

(b) The partial safety factor, γ_m, of 2.5 was to be applied.

343 JTC submitted that these assumptions were misconceived, resulting in the capacity of the brick walls being underestimated. I would now examine these assumptions in greater detail.

(1) SLENDERNESS RATIO OF THE BRICK WALL WAS EXCEEDED

344 The slenderness ratio of the brick wall (which BS5628-1 provided should not exceed 27) was obtained by taking the ratio of the effective height of the brick wall to the effective thickness of the brick wall (see also Table 7 of BS5628-1).

In summary, Jones had used a larger value for the effective height of the brick wall, and a smaller value for the effective thickness of the brick wall. Thus, when the two numbers were used in calculating the slenderness ratio of the brick wall, the result was therefore magnified.

345 On this basis, Jones contended that the slenderness ratio of the brick wall was exceeded. Dr Ting however disagreed.

346 JTC submitted that Jones' conclusion that the slenderness ratio of the brick wall was exceeded was a result of the extreme assumptions which he made in coming to his calculation of the slenderness ratio. In my view, Dr Ting's approach was more moderate and I accepted his approach.

347 Since the capacity of the brick wall was related to the slenderness ratio of that brick wall, and if Jones' assumptions were to be moderated, the capacity of the brick walls would not be as low as that advocated by Jones.

(I) EFFECTIVE THICKNESS OF THE BRICK WALL

348 Jones arrived at the axial stress limit of 1.3 N/mm^2 on the basis of an effective thickness of the brick wall of 100 mm. This would be the thickness of the brickwork alone, without taking into account the plaster / render on the same.

349 However, once the thickness of the plaster / render on the brickwork was taken into account as part of the overall thickness of the brick walls, the slenderness ratio would tend to be reduced, and correspondingly, the axial stress limit of the brick wall would be increased.

350 In fact, Jones himself calculated the axial stress limit based on the thickness of the brick walls including the applied render, in addition to two calculations of axial stress limits based on both the thickness of the bricks in the brick walls alone as well as the thickness of a doubly-thick brick wall.

351 In this regard, Jones acknowledged in cross-examination that he had measured on site (at the exposed areas) plaster or render of 15 mm on each side of the brick wall. This would put the physical brick wall thickness at around 130 mm, instead of 100 mm, although I recognised that the strength of the plaster or render was not quite the same but lower than the strength of the brickwork material itself. This would therefore mean that the “notional” or effective wall thickness to be used for slenderness calculations would not be the full 130 mm but somewhere in between 100 mm and 130 mm.

352 Further, it should be pointed out that Wyatt had used a plaster/render thickness of 20 mm on either side of the brick walls, making a total physical brick wall thickness of 140 mm.

353 When asked whether the use of 100 mm for the thickness of the brick wall was too extreme a position on slenderness, Jones' position was that in terms of the code, one should not include the render as contributing to stability, although from a practical point of view, the render *would* contribute to stability.

354 In any event, Jones agreed that a wall of 130 mm effective thickness would produce a slenderness ratio of 27 in compliance with the code, which ratio he had not previously evaluated even though he had noted from his own observations that there was plaster/render of 15 mm thick on either side of the brick walls. Jones had originally based it instead on the nominal 115 mm thickness stated in the architectural drawings.

355 As was obvious from that example, if Jones used a more moderate approach and considered the effective thickness of the wall to include the plaster / render applied to the brickwork, he would have found that the slenderness ratio of the brick wall would be reduced to the extent that it would likely comply with the code on a balance of probabilities. Most importantly, I did not find any evidence that the walls themselves had exhibited any buckling effects due to their slenderness, which indicated to me that the slenderness of the brick walls was not of any significance in contributing to the actual damage found at the Development, which were located predominantly at where the steel stiffeners were.

(II) EFFECTIVE HEIGHT OF THE BRICK WALL

356 Under section 28.3.1 of BS5628-1 (at AB 125), it was stated that the effective height of a wall might be taken as:

(a) 0.75 times the clear distance between lateral supports which provided enhanced resistance to lateral movement; or

(b) the clear distance between lateral supports which provided simple resistance to lateral movement.

357 Jones' assumption in this regard was that the effective height of the wall was the clear distance between lateral supports as only simple resistance was provided (*i.e*. the full height of the brick wall). This was Jones' basis for his arriving at the effective height of the wall of 3.05 m (for the brick wall referred to at page 335 of Jones' 1st AEIC).

358 On the other hand, Dr Ting considered that there was enhanced resistance provided, and would therefore have taken the effective height of the wall as 0.75 times the clear distance between the lateral supports. At the very least, Dr Ting would consider that the bottom of the brick wall had enhanced resistance, and that the top of the brick wall had simple resistance. I accepted Dr Ting's view that the bottom of the brick wall would have enhanced resistance against lateral movement due to the weight of the brick wall that would give rise to high frictional resistance against lateral movement. Dr Ting would not, however, be prepared to say that there was merely simple resistance at both the top and the bottom of the brick wall, unlike Jones.

359 In this regard, Dr Ting also said that the hypothetical maximum floor-to-ceiling height of 7 m of

brick walls with horizontal stiffeners breaking up the panels was inclusive of the horizontal stiffeners with depths of 0.45 m. As such, the hypothetical maximum height of each brick wall panel was some 3.275 m (= (7 - 0.45) ÷ 2) instead.

360 Thus, Dr Ting's position was that, even with simple resistance at the top of the brick wall and enhanced resistance at the bottom of the same, the hypothetical maximum height of the brick wall panel was 3.6 m (taking an average of the maximum of 3.105 m for simple resistance and 4.14 m for enhanced resistance). Hence, with JTC's requirement of stiffeners at every 3.5 m height, there would actually be some allowance left since the hypothetical maximum was 3.6 m, and Dr Ting would be happy with 3.5 m.

361 I accepted JTC's submission that Dr Ting's moderated approach was the more appropriate one to take, and the effective height of the brick wall (of a full height of 3.05 m) had to be reduced by a factor (due to the fact that the bottom of the brick wall had enhanced resistance although there was simple resistance at the top of the brick wall) thereby giving the hypothetical maximum height of the brick wall as 3.6 m.

362 In any event, even Wyatt, as the designer of the steel stiffener system, had analysed the effective height of the brick wall using 0.85 times the actual height of the brick wall, based on the Australian Code. If so, the effective height of the brick wall would be 2.59 m. Wyatt did not consider 3.5 m high walls to be too slender. In fact, he condoned locations where the height of the wall would exceed 3.5m without adding horizontal stiffeners.

363 Taking the effective height as per Wyatt (*i.e.* 2.59 m), and the effective thickness which Jones himself used (*i.e.* 100 mm) (both of which Dr Ting did not agree with), the brick wall of actual height 3.05 m would still not exceed the slenderness ratio of 27. I thus rejected the assumptions used by Jones in his calculations of the wall slenderness ratio. More importantly, I found no evidence of any defects or signs of physical distress in the walls themselves due to the walls themselves being too slender.

(III) CAPACITY REDUCTION FACTOR SHOULD BE INCREASED

364 However, the significance of the slenderness ratio was not just a matter of compliance with the code, but it made a difference to the capacity reduction factor which was used in computing axial stress limit. By using a higher slenderness ratio, the corresponding capacity reduction factor as obtained from Table 7 of BS5628-1 was reduced, and since the axial stress limit was a multiple of the capacity reduction factor, the axial stress limit calculated was also reduced.

365 Jones further reduced the capacity reduction factor by plotting his own graph for the same, justifying this on the basis that the slenderness ratio which he calculated was not even to be found in Table 7 of BS5628-1, and that this therefore allowed him to plot his own graph. This was even though in his 1st AEIC, he had simply used the lower value for the capacity reduction factor found in Table 7.

Q. For the court's benefit, if we look at the table on page 92, the table 7 capacity reduction factor from the British Standard 5628 part 1, what I understood you had done previously was to use the capacity reduction factor of 0.40 which we see against the slenderness ratio of 27?

A. Correct.

Q. What you have done on page 92 is to **<u>extend the graph</u>** at the bottom **<u>beyond the figures given by the code</u>**, so that you **<u>end up with a greater reduction factor?</u>**

A. **<u>Correct</u>**.

(Emphasis added)

366 This would obviously reduce the axial stress limit of the brick wall dramatically. In fact, Jones even prepared calculations for a brick wall panel of height of 3.5 m, and his conclusion was that the capacity reduction factor, β, was less than zero. This resulted in the axial stress limit of the brick wall being equal to zero (*i.e*. capacity of the brick wall was zero as well).

367 However, in cross-examination, Jones must have realised that this was an indefensible position, as he seemed to have quite a bit of difficulty saying that the brick wall had zero capacity (despite his calculations showing the same).

368 When told that his conclusion was that the brick wall of 3.5 m height and 100 mm thickness had zero capacity, Jones did not agree or disagree with that outright. What Jones said was that "*it doesn't comply with code at all*" and that "*this is a code compliance check*". However, when the court tried to clarify the position, Jones said that the "*code gives you zero*", prompting the Court to ask what the code gave zero for. Jones then answered "*the capacity*", but quickly added that, from a practical point of view, the tables and the code actually contained a lot of conservatism.

369 I accepted JTC's submissions that Jones' assumptions in this regard (that the brick wall was too slender, that the capacity reduction factor could be negative, etc.) were too extreme. Zero capacity was simply not consistent with reality and what was seen on site. In my view, the brick wall would have a greater capacity than what Jones gave it credit for.

<u>(IV) CONCLUSION</u>

370 To summarise, I found that the capacity of the brick wall would be greater than that calculated by Jones, who used fairly extreme assumptions in his calculations and therefore underestimated its capacity. Further, Jones overestimated the vertical movement on the brick wall, as he did not give sufficient credit to the various modes of relief of stress in the brick wall, such as the crushing of mortar and air voids.

371 After giving due consideration to some upward adjustments to the capacity of the brick wall, and some downward adjustments due to the amount of unrelieved movement, I was of the view that there was no real issue with the slenderness of the brick wall. Essentially, relief would bring the movement below 2.7 mm, and the stress would correspondingly also have decreased.

ii. Other alleged causes for the defects

372 SEC also raised various arguments why the defects in respect of which JTC was claiming were caused by and/or attributable to factors which were not within SEC's purview and/or responsibility, as follows:

(a) the contract did not specify or require movement joints at the underside of the structure where the external brickwork infill walls and stiffeners abut (paragraph 18(a) of the Defence);

(b) the thickness specified for the external wall in the contract was inadequate for the height of the external wall (paragraph 18(b) of the Defence);

(c) fair wear and tear and/or lack of maintenance of the external walls in the Development (paragraph 18(c) of the Defence).

373 SEC belatedly filed further Further and Better Particulars of its Defence on 5 July 2007 (just prior to trial) to include a reference to a powerpoint presentation prepared by JCPL in 2005, in response to JTC's request for particulars as to the factors which the defects observed could have been caused by and/or attributable to and/or contributed by.

374 However, it was evident from the cross-examination of SEC's witnesses (and in particular, its expert) that certain points raised in the JCPL powerpoint presentation were merely latched on to by SEC as an afterthought, and that even SEC's witnesses did not seriously believe in the same as a cause of the defects observed.

375 In this regard, when Jones was asked in cross-examination whether, on a general level, he was endorsing the views that were put forward by JCPL or whether he was merely noting that certain things had been said by JCPL, Jones' response was that it was a combination of the two. Jones then clarified that he believed that JCPL was correct in identifying the structure as being complex and behaving significantly in the X and Y directions, subject to lateral load.

376 However when cross-examined on the specifics, Jones appeared to be echoing the comments made by JCPL on a "qualitative" (and, selective) basis. As expected, when JCPL's comments were adverse to SEC (*e.g.* in identifying the stiffeners as a possible cause of the defects), SEC was not prepared to accept them.

a. Were the defects caused by the absence of movement joints?

377 SEC's central contention in its pleaded Defence was that the defects observed could be caused by and/or were attributable to and/or were contributed by the fact that the contract "*does not specify or require movement joints at the underside of the structure where the external brickwork infill walls and stiffeners abut*" (see paragraph 18(a) of the Defence).

378 Specifically, SEC said that horizontal compressible joints at each floor level between the structure and the brick walls ought to have been provided by JTC. However, in their absence, when coupled with shrinkage and creep of the r.c. frame, and expansion of bricks (see Jones' 1st AEIC, *e.g.*, at paragraph 10.2, page 47), the movement of the r.c. frame and the growth of the bricks caused the defects observed on site.

379 Thus, Jones suggested that even if the originally specified r.c. stiffeners had been used instead of steel stiffeners, there would still be bowing and cracking. As such, the defects observed on site were not due to any shortcoming in the design or construction of the steel stiffeners.

380 In other words, SEC was contending that:

(a) if horizontal compressible joints had been provided, the defects observed would have not occurred. As such, the steel stiffeners did not cause the defects observed on site;

(b) if r.c. stiffeners had been used, the defects observed on site would still have occurred as horizontal compressible joints were not provided – that was why bowing and cracking of the brick walls were still observed at areas at the Development where r.c. stiffeners were located. (This point shall be dealt with in greater detail below.)

381 However, SEC faced the preliminary hurdle which was to prove that it was JTC's responsibility to design and provide these horizontal compressible joints and not SEC's (assuming that horizontal compressible joints were required in the first place). In my judgment, SEC would fail on this defence in this respect alone. In my view, it was clearly SEC's responsibility to provide these horizontal compressible joints, if they were indeed necessary to enable the steel stiffener system to perform satisfactorily and to be fit for its purpose.

1. IF MOVEMENT JOINTS WERE PROVIDED, DEFECTS IN THE VERTICAL STEEL STIFFENEERS WOULD STILL HAVE OCCURRED

382 As explained above, even if movement joints had been provided for in the Contract for the horizontal stiffeners, no movement joint would have been provided in any case where the vertical stiffeners connected to the floor slab above. There would be no issue had the stiffeners been made of r.c., but with vertical steel stiffeners (which would not shrink and creep with the r.c. structure), the compressive loads (on Jones' own account) would still have resulted in defects. In other words, provision of horizontal compressible joints would not have alleviated the compressive loads on the vertical steel stiffeners which were designed to be welded to the stubs that were bolted to the r.c. structure. The vertical steel stiffeners would have failed under axial load in any event, and SEC would have remained liable to JTC for failing to provide steel stiffeners that were fit for use in the Development.

383 It was obviously not for JTC to stipulate how the vertical steel stiffeners should be connected to the floor slab above. That connection was designed by WP Brown, whom SEC had engaged.

384 At the brick wall panels (*i.e.* not at the vertical steel stiffeners), JTC said that the 2.7 mm vertical movement would not have occurred as there would have been relief provided by the soft mortar joints crushing, the crushing of the air voids in the mortar, the lateral deflection of the brick wall panels, the vertically-placed/flipped bricks and some accommodation by the horizontal steel stiffeners.

385 According to Dr Ting, the primary cause of the defects was the low lateral stiffness of the steel stiffeners under lateral load. Even if horizontal compressible joints had been provided at the top of the brick wall panel underneath the floor slab, the defects would still have been caused. I accepted Dr Ting's opinion that the low lateral stiffness of the steel stiffeners under lateral load was the primary cause of the defects. In my view, even if it were true that there was some vertical movement in the r.c. structure of some 2.7 mm (with part of that vertical movement relieved

accordingly as explained earlier), at best that could only constitute a secondary contributory cause of the defects found at the site.

2. IF R.C. HAD BEEN USED, NO DEFECTS WOULD HAVE BEEN OBSERVED

386 In this regard, the evidence of Sim (the key witness from SEC) was clear; speaking from experience, Sim himself was of the opinion that horizontal compressible joints were usually not provided in brick walls with r.c. stiffeners, and this would not have caused any problem.

> Q: As an engineer, you know that RC beams and slabs are expected to deflect under loading as well, correct?
>
> A: Correct.
>
> Q: Would it not be a problem to pack the bricks and mortar tightly up to the bottom of the beam or slab, so either you end up with the brickwall taking the deflection, so either you end up with the brickwall taking then deflection, or you are trying to stop the deflection altogether?
>
> A: **Normally for all the other construction projects that I have done, this was the normal practice that we have done. It doesn't have that problem**.
>
> Q: So you would expect the contractor to leave a sort of 10 mm joint or gap at the top course of bricks and fill that with mortar, right? **Your experience, Mr Sim, is that brickwalls constructed like that built up to the bottom of the beam or slab do not have problems in practice, correct?**
>
> A: **Correct**.
>
> (Emphasis added)

387 I accepted the evidence of Sim that based on his experience at other construction projects, if r.c. stiffeners had been used instead of steel and with packing of the bricks and mortar up to the bottom of the beam or slab, there would not have been the same defects observed at all. Furthermore, JTC would not have to incur substantial costs in rectifying the defects as compared to the rectification of the steel stiffeners.

b. Defects were not caused by slenderness of the brick walls (re paragraph 18(b) of the Defence)

388 As stated above, "slenderness" of the brick walls was said by SEC to be a contributory factor for the defects observed; more specifically, SEC said that the thickness specified for the external wall in the contract was inadequate for the height of the external wall. The walls were alleged by SEC to be more prone to cracking and ultimately to buckling.

389 Dr Ting rejected the assertion that the external walls were too slender. Further, he pointed out that the mode of failure/distress of the brick walls did not indicate a wall thickness that was inadequate, which would result in compression failure. I accepted that the defects found at the Development did not evidence any failure or distress of the brick walls due to inadequate wall

thickness.

390 The reasons why SEC's / Jones' assertion that the brick walls were too slender was unfounded have been set out at [348] onwards above. JTC said that the brick walls were not too slender, and the thickness specified for the external walls was in fact adequate for the height of the external walls.

391 This seemed also to be Wyatt's opinion at the material time in 1998 or 1999, as he did not think then that the JTC spacing requirements resulted in walls that were too high or too slender. They were "*not so far out of line that [he] was concerned*".

392 In cross-examination when Jones was describing the horizontal steel stiffener as the "*weakest link*" in the packing scenario, Jones' conclusion was that, as the steel stiffener had a capacity lower than the brickwork above and below the horizontal steel stiffener, and when the set-up was loaded up (as it were), the horizontal steel stiffener would experience side-sway deformation, even before the full load induced by the creep and shrinkage of the r.c. frame and the expansion of the brickwork could materialise.

393 In that event, the bowing of the walls due to the induced loads would not even come into play, as the horizontal steel stiffener would first side-sway as the brickwork tried to expand.

A.	The first thing, your Honour, is the calculation of what happens to the horizontal stiffener embedded in the wall when you have brickwork both above and below, you have the packing case and the effect that would have on the bow of the wall, bearing in mind you would get side sway and vertical compression of the stiffener. The actual -- from the panel above the stiffener, the actual load generated by the 2.7 mm compression would be 620 kilonewtons. For the panel below the stiffener, the load generated would be 450 kilonewtons from the expansion of that panel as well as the shrinkage and creep acting from below effectively pushing up. The steel horizontal stiffener only has a capacity, based on Dr Chiew's tests, of 64 kilonewtons, so what I think would happen is that as you approach 64 kilonewtons, you would get relaxation of the shrinkage, creep and expansion loads, such that bow does not occur, and you'd get side sway and vertical movement of the steel stiffener, which is what you are actually seeing on site. So the horizontal steel stiffener is compressing, swaying sideways, before the big creep and shrinkage load and the expansion of brickwork load can be generated, so it relaxes. So the **weakest point in the system is the steel stiffener...**
…	
	Well, the conclusion is -- my conclusion is still the same, that the distortion of the horizontal stiffeners is caused by this load.
COURT:	By the creep?
A.	Shrinkage and creep and the brick expansion.

COURT: Which couldn't reach it but forced it out?

A. Well, it's actually the load imparted --

COURT: Forced the lateral side sway?

A. Yes. The load imparted on the steel stiffener is sufficient to cause that side sway.

COURT: There is no need for a bow; the bow doesn't even reach?

A. The bow doesn't even come into play with the steel --

COURT: So the wall remains straight, it is just pushed out as it tries to expand?

A. **<u>Yes</u>**, your Honour...

(Emphasis added)

394 Accordingly, Jones agreed that the more likely shape which the construct (of the brick wall above, followed by horizontal steel stiffener, followed by brick wall below) would take would be a **<u>Z-shape</u>** rather than that countenanced in "Case 2A" and "Case 2B" as drawn at page 2 of D-43. This was explained to be due to "***<u>the stiffener itself is deforming laterally in side sway, that would tend to force the wall to deform more into the Z shape"</u>***. Further, as the upper and lower walls were not necessarily of equal height and as the wind forces would not necessarily be uniform over the whole wall, the lateral forces imposed by each wall on the upper and lower flanges of the horizontal steel stiffeners would not likely be the same. The differential lateral forces on the upper and lower flanges would also in my view tend to force the horizontal steel stiffeners to take a Z-shape configuration under the side-sway deformation rather than the "Case 2A" and "Case 2B" configuration as drawn at page 2 of D-43. The visual inspection of the deformed horizontal steel stiffeners at the Development also tended to bear out the Z-shape deformation.

395 An important point to note would be that although arguably in the "Case 2A" and "Case 2 B" configuration there might not be an induced torsional force in the horizontal steel stiffeners as submitted by SEC, however in the more likely Z-shape deformation, there would certainly be torsional forces acting on the horizontal steel stiffeners. The weak torsional rigidity of the completely hollow horizontal steel stiffeners (which were prone to side-sway as there were no stiffened cross-sectional internal bulkheads to prevent distortion of the cross-section and prevent the side-sway) as compared with the solid r.c. stiffeners (which would have much better torsional rigidity) had, in my assessment, aggravated the extent of the defects and damages observed at the Development.

396 As the horizontal steel stiffener would side-sway before the brick walls above and below it could even bow, that would also indicate that "slenderness" of the brick walls was not what caused the defects. (At this juncture, it would be pertinent to note that Jones' position was not that the brick walls were too slender under lateral load.)

397 As such, despite SEC's pleading that the slenderness of the brick walls was a cause of the defects observed (by paragraph 18(b) of the Defence), its own expert was not prepared to support

that assertion put forth by SEC.

398 Indeed, what SEC’s own expert said was that the horizontal steel stiffeners were the “***weakest link***” in the packing construct, and therefore they deformed laterally in side-sway, forcing the brick walls to deform into a Z-shape. Simply put, SEC’s own expert’s evidence in this regard actually supported JTC’s claim that the steel stiffeners were not fit for their intended purpose.

c. Defects were not caused by fair wear and tear and/or lack of maintenance of the brick walls (re paragraph 18(c) of the Defence)

399 SEC had originally asserted that fair wear and tear and/or lack of maintenance of the external walls in the Development was a cause of the defects observed. However, it would appear from the face of the AEICs filed by SEC's witnesses that SEC had dropped this assertion.

400 Neither was this assertion pursued with Dr Ting, JTC's expert, notwithstanding Dr Ting had specifically rebutted SEC's assertion in this regard in his report (Dr Ting's AEIC, "TSK-2" at paragraph 8.1(f)).

d. Defects were not caused by weakness in the building structure

401 As mentioned above, SEC had obtained leave to file further Further and Better Particulars of the Defence at the eleventh-hour, to rely on the JCPL powerpoint presentation.

402 Jones had relied on the JCPL powerpoint presentation in his 1st AEIC to say that the “*conclusion by JCPL clearly states that JTC’s structural design is the primary cause of the defects to the brickwork infill walls and stiffeners, and that the steel stiffener design was satisfactory under the as-designed condition*”.

403 Jones also relied on the JCPL Stage (I) Investigation Report ("JCPL (I)") (at AB 4339) selectively, and adopted certain assertions made by JCPL in those reports. However, Jones never referred to JCPL Stage (II) Investigation Report ("JCPL (II)") (at AB 8368), which qualified the assertions made by JCPL in JCPL (I).

404 I observed that JCPL (I), JCPL (II) and the JCPL powerpoint presentation never had the benefit of a full quantitative analysis.

405 JTC's witness, Ms Cherie Kee, had given evidence that the JCPL powerpoint presentation was presented by JCPL to JTC on or around 22 November 2004, and that she was present at that presentation. As far as she could recall, the JCPL powerpoint presentation did not contain any formal conclusions, because there was supposed to be a further in-depth or a detailed investigation carried out. As such, the JCPL powerpoint presentation only raised the possibility of various causes.

406 That this qualitative assessment carried out by JCPL was a preliminary one (and not a thorough, detailed analysis) might also be the reason why there might seem to be contradictory statements made by JCPL within the same powerpoint presentation, such as the statements at AB 605 and 619.

COURT: Ms Kee, just a few clarification questions. At page 605, you notice under point 3, on the upper slide, it is stated that: "The steel stiffener did not fall short from design requirement ..." -- and then at page 619, they say the steel stiffeners were one of the causes to the brickwall defects. Did anyone try to reconcile these two things?

A. I don't recall there was any discussions on the two different points.

407 Similarly, JCPL (I) was admittedly a qualitative assessment as well. As stated by JCPL, "*[t]he building structure was examined* _*qualitatively*_ *by reviewing the structural rigidity of the frame, individual member capacity and adopted construction detailing*" (emphasis added).

408 Despite all these indications that the JCPL documents were prepared on the basis of preliminary, qualitative assessments only, SEC and/or Jones still sought to rely on them for assertions that there were other causes of the defects observed (apart from the steel stiffeners installed). However, I noted that many of these points were, however, abandoned or not pursued at trial.

1. ALLEGED ROTATIONAL EFFECT OF BUILDING STRUCTURE

409 Initially, Jones appeared to endorse the "*structural inadequacy*" of the design of the building structure (and the alleged resulting "*twisting and deformation of the building structure*") as the "_*main cause*_ *of the defects*" (emphasis added). However, Jones subsequently admitted that his analysis in respect of this purported "*cause*" was one carried out on a purely qualitative basis.

410 JTC submitted it was difficult to believe that Jones had, in the first place, found himself to be in a position to agree with the speculation that structural inadequacy could be the "*main cause of the defects*", without either JCPL or himself carrying out a proper, detailed quantitative analysis in that respect.

411 In any event, Jones was asked about page 45 of his 1st AEIC, where he commented on various documents. Jones first said that he did not believe that there was deformation or deterioration of the primary structure of the building to date, but he did believe that there was such deformation or deterioration of the secondary structure of the building. In this regard, Jones specifically clarified that by "*primary structure*", he meant the primary columns and beams, and by "*secondary structure*", he meant the brick walls and stiffener system.

412 However, immediately after that exchange, when Jones was referred to paragraph 9.3 on page 45, and in the light of that exchange, Jones was asked whether when he talked about deformation in paragraph 9.3.1, he was talking about the deformation of the secondary structure of stiffeners and walls, Jones contradicted himself by saying that he was talking about the deformation of the primary structure, which was then interacting with the secondary structure.

413 Jones was then asked to explain what deformation of the primary structure he was referring to. After some silence, Jones gave a convoluted explanation to the effect that there was some rotational effect of the primary structure. However, Jones never calculated these deflections of these rotational effects, and Jones' "*assessment*" in this respect was merely "*qualitative*". Jones further added that "*[a]ll we have is the analysis undertaken by JCPL*". It did not appear to me that there was much of a proper analysis, let alone a 3D (three dimensional) analysis of the building structure, undertaken by

JCPL that I could discern from the evidence before me. I thus gave little weight to Jones' comments that stemmed from his reliance on the JCPL report.

2. HORIZONTAL CRACKS IN COLUMNS NOT INDICATIONS OF STRUCTURAL INADEQUACY

414 In this regard, Jones had commented (at paragraph 8.0 of his 1st AEIC, under the heading "Para 6", part (b), "Structural design of main building", at page 43) that:

> "Note that this section of the JTC report is an **admission of structural inadequacy** arising from defective design as demonstrated by **horizontal cracks in the columns**. Such cracking is likely to be a result of floor beam post-tensioning causing deformation and additional moments in the columns due to shortening of the beams **under compressive load and creep**, combined with asymmetric structural layout leading to twisting of the floor plates relative to each other under normal loading resulting in column cracking. **This deformation, or movement, will also induce stress in the brickwork infill walls resulting in crushing and bowing of the brickwork as evidenced at the Property."**
>
> (Emphasis added.)

415 In any event, under cross-examination, Jones agreed that the horizontal cracks in the columns were not an indication of compressive loads applied vertically; Jones also did not believe that these cracks observed had anything to do with the axial loads or the structural framework being too flexible.

416 All in all, under cross-examination, Jones distanced himself from the position taken in his 1st AEIC, and rejected the notion that these observations of horizontal cracks in columns were even a major structural issue at all.

A.	Yes. I don't believe it's anything to do with the axial loads because the axial load would tend to press.
COURT:	Press down.
A.	And the cracks close again --
COURT:	Evenly, so you shouldn't get this, unless of course the pillars themselves move out.
A.	But these are very massive.
COURT:	So it's not likely to be due to that?
A.	It's not likely to be due to that, so what this is telling me is that this bending is having a greater effect than the axial load at these locations. It's --
COURT:	It's nothing to do with the framework being too soft?
A.	Flexible, yes, your Honour. "Soft" is a very emotive word for engineers.

COURT: Yes, too flexible. <u>So it's nothing to do with flexibility?</u>

A. **<u>Yes</u>**.

COURT: Do you mean to say that the effects due to the post-tensioning differential are more than that due to the axial load?

A. Yes, your Honour, at this stage…Then the axial load builds up after. So my own view is probably that these cracks occurred, they were wider when they started with this process and the axial load has expressed them but you are still left with a hairline crack. **I don't believe they are a major structural issue. It just explains what --**

COURT: It just explains the phenomena.

A. -- what they are doing there.

(Emphasis added)

417 As such, despite what Jones had said in his 1st AEIC, it was clear that he had moved away from the proposition that horizontal cracks in the columns indicated structural inadequacy in the building.

3. ALLEGED "SOFT COLUMN"

418 JCPL's powerpoint presentation listed "*'soft' column*" as one of the "*causes to the brick wall defects*". Dr Ting explained substantively why the JCPL powerpoint presentation was inaccurate. I had no reason to reject his explanation.

419 Jones admitted that he had not done his own analysis to check what JCPL had asserted. Without proper analysis, Jones should not have aligned himself with JCPL's comment that "*soft column*" was a possible cause of the defects observed. It would appear that Jones simply adopted JCPL's conjectures. Indeed, JCPL had not carried out a soft column check, but had instead only recommended that this be done.

420 Further, Jones had taken the position during cross-examination that the "*soft column*" was not the actual cause of the defects. Further, when asked by the Court whether it was a significant cause, Jones said that he believed it was a local cause only, and would not be a significant cause.

MR MANIAM: Mr Jones, look at page 44, next to the reference to 7B, soft column. I have to ask you, Mr Jones, whether you considered this to be a cause of the defects in this case?

A. I believe it has contributed, <u>but not the actual cause</u>.

COURT: Is it a significant contribution or a fairly insignificant contribution?

A.	When looking at the effects of shrinkage and creep in a concrete building, it's actually related to the stress in the columns. So, if you have got a small column carrying a relatively high load, it will tend to creep more than a large column carrying a lower load. So, you'll get a differential shrinkage and creep, which I believe is what we have got with this soft column.
COURT:	Whether it's a significant cause of the cracks all over the building?
A.	I would believe **<u>it's a local cause only</u>**.
COURT:	A local cause. It won't be a significant cause of (unclear) because some of them are fairly far away.
A.	**<u>Correct</u>**, your Honour.

(Emphasis added)

421 If Jones was seeking to rely on the JCPL Investigation Reports, he ought to also have cited JCPL (II) in conjunction with JCPL (I). Instead, he picked and chose the portions of the JCPL Investigation Reports that were advantageous for SEC.

422 In JCPL (II), under paragraph 8.2 "Column capacity" (see AB 8377), JCPL stated that the column capacity was not related to the defects in the vicinity of the steel stiffeners. The relevant parts of paragraph 8.2 are reproduced herein as follows:

> However there is **<u>no observation of column cracks on the reinforced concrete columns</u>** at the time of the visual inspection. It was believed that the column has not been subjected to the full design loads since there were vacant or partial vacant units. As such, the capacity of the column is not related to the defects in the vicinity of the steel stiffeners/lintels.
>
> It was noted that the brick wall construction carried out at the vicinity of this under provided column had considerable workmanship shortfalls.

(Emphasis added)

423 As Jones had noted, the periodic inspection report dated 9 May 2005 found no structural defects, deformation or deterioration in the building structure. It was clear that JCPL's assertions as stated in the powerpoint presentation were not based on proper detailed analysis. Hence, it was not appropriate for Jones to have relied on these untested assertions as his considered expert opinion on the cause of the defects.

424 In the light of the above, I did not find anything wrong with the building structure, and the assertions of the "*rotation of building*" and "*soft column*" as causes of the defects observed were no more than red herrings. SEC's assertion that "*the technical evidence clearly suggests that there is a fundamental problem with the structure of the Development*" was without basis and was premised on its wholesale adoption of the preliminary views of JCPL, which Jones had not verified by any quantitative assessment. I thus rejected SEC's contention that the defects had been caused by JTC's

irregular design of the Development structures.

D. R.c. Stiffeners Better Than Steel

i. No fully developed r.c. design

425 It was common ground that there was no fully-developed r.c. design, and the parties eventually compared Wyatt's steel stiffener design against certain sketches in respect of r.c. stiffeners and tie-beams.

426 In this regard, JTC submitted that:

(a) if the r.c. stiffeners were in fact built on site, there would have been a fully-developed r.c. stiffener design (rather than the simplistic sketches used in the parties' comparison of r.c. design versus steel design); and

(b) in any event, the sketches which the parties were analysing for the purposes of comparison were not meant for a stiffener system for use in brick walls generally as such.

a. JTC would have constructed on basis of a fully-developed r.c. stiffener design

1. NO DESIGN REQUIREMENTS FOR R.C. GIVEN

427 JTC rightly pointed out that there were no design requirements for the r.c. stiffener due to the substitution of steel – the only requirement as such would have been that the r.c. stiffener, if built, would have to have a compatible width with the width of the brick wall that it was to form a part of.

428 Despite this, it was suggested to Gary Ng in cross-examination that "*the JTC drawings and specifications dictate the requirement for the RC lintels and stiffeners for the brickwalls*", and that there was no approval for the r.c. stiffener design because "*everything is already set in the JTC specifications and drawings*". Gary Ng disagreed with these suggestions.

429 Gary Ng's evidence in this regard was that there was no design of the stiffeners for the brick wall, and only the intervals and heights of the stiffeners (*i.e.* the spacing requirements) were provided. This would therefore only dictate the width of the stiffener to be used, because it had to be flush with the brick wall. The width of the stiffener could be 130 to 140 mm if the architect did not mind the stiffener sticking out. But for the height of the stiffener (*i.e.* the depth), it would still depend on the load.

430 Accordingly, for brick wall stiffeners, JTC did not even provide the size and dimensions of the same, but only provided the loading details.

431 In any event, Gary Ng's evidence was that, even if the material remained as r.c., he would consider SEC to be responsible for the design of the entire system of brick wall stiffeners, and would insist on a design endorsed by a Professional Engineer in this regard.

432 As such, SEC could not simply shirk its responsibility to JTC in respect of providing a functioning steel stiffener system by suggesting that since the equivalent r.c. stiffener design was contained in

the JTC specifications and drawings, it implied that JTC somehow was responsible or partly responsible for the substituted steel stiffener design proposed by SEC.

(I) JTC INTENDED END CONDITIONS OF R.C. TO BE FIXED

433 Jones said in his 1st AEIC that:

> The reinforced concrete horizontal tie beams are fixed **monolithically** to the reinforced concrete columns…
>
> (Emphasis added.)
>
> The JTC design calls for reinforced concrete vertical and horizontal stiffeners to support the brickwork walls. The JTC stiffener details do not indicate the connection or interface details with the horizontal and vertical reinforced concrete structure. Where constructed, the **conforming detail adopted and approved on site by JTC is a monolithic connection** without any movement capability…
>
> (Emphasis added.)

434 When asked whether the r.c. tie-beams (connected to r.c. columns) on site had fixed connections, Jones was again reluctant to give a straight answer, but in the course of cross-examination, was understandably hard-pressed to answer in the negative.

Q.	And so, as far as the connection between reinforced concrete tie beams and columns is concerned, it has been possible to achieve a fixed connection?
A.	The tie beams in question are very large beams. They are so deep in respect of the span, I do not believe they need to develop any end fixity. They are very rigid elements in their own right. In fact, the stress in those beams would be very light, as one looks at the ratio of the depth of the beam to the span of the beam.
Q.	In any case, for the purposes of analysis, you would be prepared to treat them as fixed end connections.
A.	It would be unusual to treat them as fixed end connections in design because they are secondary elements.
COURT:	These RC tie beams -- your columns in the question. Do you refer to the actual RC tie beams and RC columns?
MR MANIAM:	What is currently on site, your Honour, which your Honour has --
COURT:	Excluding the steel ones, these are the structural tie beams and structural RC columns?
MR MANIAM:	That's right.

COURT:	And you're asking whether or not they -- between them -- amongst themselves have achieved fixed connections?
MR MANIAM:	That's right, your Honour.
COURT:	So his answer is? No?
A.	My answer is there would be no need for them to develop --
COURT:	-- (unclear -- simultaneous speakers) -- a question of whether there is in this construction, of whether there's -- he's not asking whether there's a need -- asking whether they have achieved it through the method of construction.
A.	Well, the beam is so deep that it will achieve fixity very easily --
COURT:	That's what the question is all about, whether it has achieved that structure, and the answer is "yes".

(Emphasis added.)

435 As such, it was clear that for the r.c. tie-beams and columns at least, it was possible for the connections to be fixed, and this was in fact the case.

436 Even for r.c. stiffeners (although only constructed in certain specific locations), JTC would have intended the end connections to be fixed in the fully-developed r.c. stiffener design. Jones agreed that for the horizontal r.c. stiffener at least, the connection could be a monolithic one.

437 Jones then tried to reject the possibility that JTC would have intended to achieve a monolithic connection between the r.c. stiffeners and the r.c. structure, by saying that if that was the case, the design engineer would have included on his drawings such a detail, and that the fact that "*JTC's design sketches produced for construction do not show end connections, either for horizontal or for vertical stiffeners indicates to [him] that their intention is to have pin connections for purposes of design*".

438 It was crystal clear that Jones was really trying to defend an indefensible position. The undisputed fact was that there was no fully-developed JTC design produced for construction of the hypothetical r.c. stiffeners in the first place.

439 In fact, when questioned further by the Court, Jones said that he could not answer the question, because he only had cross-section details. This, coupled with the fact that the sketches were not meant for the general stiffener system in the first place, was consistent with JTC's intention for fixed end connections in a fully-developed r.c. stiffener design (which was not present in this case).

COURT:	Okay. So there are two questions there, based on drawings firstly, for those that have actually been constructed out of RC stiffeners, do you agree whether they are fully fixed, or they are partially fixed or what? Second question is, as-built, are they fully fixed, partially fixed or pinned?
A.	I think that is the question, but I don't think I can answer that.
COURT:	No.
A.	Because, (a), **<u>you have only cross-section details</u>** provided by JTC, either with, for example, four bars such as the 4T-16s or the two bars, 2T-25s.

(Emphasis added)

440 Further, the drawings for the MB1 r.c. stiffeners, as well as P-18, showed fixed end connections, which would indicate that JTC would have intended fixed end connections for design, if indeed the r.c. stiffeners were constructed (rather than steel stiffeners).

441 In any event, JTC's position was that JTC would have wanted a fully fixed end connection, had the r.c. stiffeners in fact been built. Further, had the r.c. stiffeners in fact been built, they would have been better than the steel stiffeners in all material aspects.

442 As it stood, the r.c. design as per Gary Ng's sketches were already better than the steel design, even though it was merely a "lower bound" of sorts, since the r.c. design, if fully developed, would have been better in my assessment.

b. Sketches were not meant for use in brick walls generally

443 The starting point in this regard would be the Contract, which had an r.c. stiffener/lintel detail contained in the standard specifications for civil/structural works. However, this detail was only for r.c. stiffeners/lintels of spans up to 2.4 m, and was silent on r.c. stiffeners/lintels of spans greater than 2.4 m.

As such, the parties focused on certain sketches of the r.c. design in the comparison against the steel design. It was not in dispute that the sketches (of 4T16 detail for the r.c. stiffeners) were prepared by Gary Ng (the Resident Engineer for the Project), who was not a Professional Engineer at the time.

444 However, Gary Ng's evidence was that these sketches were intended for spans over openings, and the stiffeners in the sketches were "*supposed to be tying across columns or columns where there is a window area or where there is a door area, or where there is a hole*". Further, Gary Ng was adamant that the sketches "*are in no way connected to the details as specified by architect on the brickwall stiffeners*".

445 As such, the parties were not quite comparing apples with apples in using these sketches of r.c. design in a comparison against Wyatt's steel stiffener design, since a proper comparison should only be undertaken with a fully-developed r.c. design prepared for the same purpose as the steel

stiffeners.

446 In any event, Gary Ng also gave evidence to say that the 125 mm by 450 mm (4T16) detail in the sketch which he provided for lintels were meant for spans not exceeding 3 m; should the span of the lintel exceed 3 m, Gary Ng (as the person who prepared the sketch) would expect SEC to seek verification for the same. In any event, the sketches prepared by Gary Ng were subsequently revised.

447 Leaving aside the absence of a fully-developed r.c. stiffener design, the steel stiffeners still fell short of the mark even on the basis of the simplistic sketches of the r.c. stiffeners. The steel stiffeners were thus not equal to or better than the r.c stiffeners that should have been provided.

448 In the following sections, I will deal with the comparison between r.c. and steel in greater detail.

ii. Steel vs r.c.

a. R.c. stiffeners would perform better than steel stiffeners under lateral load

449 JTC's expert, Dr Ting, considered that the unsatisfactory performance of the steel stiffeners under lateral load was the primary cause of the defects which were observed on site. A corollary to that contention was that the hypothetical r.c. stiffeners would have performed better than the steel stiffeners in this regard.

450 As such, Dr Ting carried out certain calculations (see P-38), and found that the lateral deflection under lateral load (with reference to the design wind load of 0.5 N/mm^2) for the vertical steel stiffener (V1-200) in a 7 x 10 m hypothetical set-up would be around 147.2 mm (with a pinned-pinned end condition), while the lateral deflection under lateral load for the vertical r.c. stiffener would be around 7.8 mm (with fixed-fixed end condition, uncracked section).

451 This would translate into a span/48 deflection ratio for the vertical steel stiffener, which would not comply with the code (of span/360) according to Dr Ting; according to Jones' standards (of span/150 to span/180), a deflection of this magnitude would result in damage to the plaster/render already.

452 On the other hand, the calculated lateral deflection of the vertical r.c. stiffener under lateral load (of 7.8 mm over a span of 7m) would translate into a span/897 deflection ratio, which would amply comply with the standards which either expert said would apply. In terms of practical consequences, this would mean that no damage would be caused to the plaster/render as a result of this very small deflection of 7.8 mm had the r.c. stiffener system been used. In this regard, it should be noted that the permissible deflection ratio for the r.c. stiffeners (span/250 from BS8110) was less stringent than that for the steel stiffeners (span/360 from BS5950). The r.c. stiffeners, with significantly lesser deflection in absolute terms as well as relative to code requirements, would therefore actually outperform the steel stiffeners by a significant extent.

453 As such, it was apparent on the face of Dr Ting's calculations that the lateral deflection of the vertical steel stiffener was significantly greater than that of the vertical r.c. stiffener under the same lateral load (and hence the deflection ratio was significantly lower for the vertical r.c. stiffener).

454 This was also true for the other four brick wall configurations presented in Dr Ting's table at page 1 of P-38. In all the brick wall configurations analysed, the vertical r.c. stiffeners performed better than the vertical steel stiffeners.

455 Wyatt, as the designer of the steel stiffener system, had also given evidence in respect of the lateral deflections calculated by Dr Ting. In this regard, although Wyatt considered that the values for the deflections under lateral load for the horizontal r.c. stiffeners (as contained in Table 5 of P-24) were somewhat less than what he had anticipated them to be, he was nonetheless of the view that for the H1-150 and H1-250 horizontal steel stiffeners at least, the deflections would still be greater than that for r.c., and only for the H1-300 horizontal steel stiffeners would the results have been comparable.

456 This was also the case on Jones' calculations as well. In D-29 prepared by Jones (at page 3, Table of Results), Jones presented a comparison of the vertical and horizontal members under lateral load and vertical load (in the hypothetical 7 x 10 m set-up), reproduced for ease of reference as follows:

Member	**Material**	**End Condition**	**Max Deflection δ, mm**	**Remarks**
Lateral Load				
Vertical Stiffener 125 x 200 (4T16)	RC	Fix - Fix	71.3	Ultimate failure in both cases
		Fix - Pin	149.5	
Vertical Stiffener 125 x 300 (2T25)	RC	Fix - Pin	150.4	Ultimate failure in Stiffener
V1-200	Steel	Pin - Pin	156	Stiffener OK
Horizontal Stiffener 125 x 450 (4T16)	RC	Pin - Pin	24.3	Beam OK
Horizontal Stiffener 125 x 450 (2T25)	RC	Pin - Pin	28.9	Beam OK
H1-300	Steel	Pin - Pin	16.5	Beam OK
Vertical Load				
Horizontal Stiffener 125 x 450 (4T16)	RC	Fix - Fix	1.6	Beam OK
Horizontal Stiffener 125 x 450 (2T25)	RC	Fix - Fix	1.6	Beam OK

H1-300	Steel	Pin - Pin	14.8	Beam OK
		Fix - Fix	13	Beam OK
For the horizontal stiffeners, it is important to note that additional Δ due to eccentricity is negligible as critical member is the vertical stiffener. Δ due to eccentricity would be accounted for in case of combined axial load and bending in lower half of vertical.				

457 On the face of the above table, the maximum deflection which Jones had calculated for both types (with fixed-pinned end conditions for the r.c. stiffeners, and pinned-pinned end conditions for the steel stiffeners) were about the same in magnitude (of around 150 mm for the vertical r.c. stiffeners and around 156 mm for the V1-200).

458 However, if a fixed-fixed end condition was assumed for the vertical r.c. stiffener, the maximum deflection calculated by Jones for the vertical r.c. stiffener would be about half of the deflection when a fixed-pinned end condition was assumed - of around 71.3 mm (for the 4T16 r.c. stiffener).

459 As would be apparent from this comparison, the disparity between Dr Ting's calculations (of 7.8 mm deflection for fixed-fixed end condition) and Jones' calculations (of 71.3 mm deflection for fixed-fixed end condition) for the r.c. stiffeners was significant. Dr Ting's calculations were on the basis of a composite analysis of the vertical and horizontal stiffeners interacting together as a stiffening frame in the same brick wall set-up. On the other hand, Jones' calculations were on the basis of the horizontal r.c. stiffener and vertical r.c. stiffener acting independently, which was unrealistic. Jones had not carried out the calculations on the basis of a composite analysis of the vertical and horizontal stiffeners acting together as a stiffening frame in the same brick wall set-up, which in my view he ought to have done.

1. COMPOSITE ANALYSIS OF DEFLECTION

460 Jones did not carry out a composite analysis of the vertical and horizontal r.c. stiffeners acting under lateral load. Instead, he calculated the lateral deflection of the vertical r.c. stiffeners and horizontal r.c. stiffeners separately for each individual member.

461 If he had calculated the lateral deflection of the vertical and horizontal stiffeners under a composite analysis (as Dr Ting had done), the resultant lateral deflection of the steel stiffeners would be significantly greater than the r.c. stiffeners.

462 This was because r.c. stiffeners would act together under load such that the horizontal r.c. stiffener would curb the deflection of the vertical r.c. stiffener when the latter deflected under lateral load as there would be continuity in both directions in the case of the r.c. stiffener system and the movement would be minimised.

463 On the other hand, for the steel stiffeners, the lateral deflection of the vertical steel stiffener would move the intersection with the horizontal steel stiffener by the amount of that deflection. The horizontal steel stiffener would not act to reduce that deflection, as the connection of the horizontal stiffener to the vertical stiffener was essentially pinned.

464 As such, even though it would appear (superficially) that the deflections calculated by Jones

for the horizontal r.c. stiffeners (of 24.3 mm for 4T16, and 28.9 mm for 2T25) were greater than the deflections calculated for the H1-300 horizontal steel stiffeners (of 16.5 mm), that was in essence due to Jones not having analysed the deflections with the vertical and horizontal r.c. stiffeners interacting together.

465 I found that Jones' approach was erroneous as the behaviour of an r.c. stiffener system was different from that of a steel stiffener system because of the difference in the end conditions of the stiffeners. The correct and more realistic approach in my view was that adopted by Dr Ting.

2. END CONDITIONS OF THE STIFFENERS

(I) EXPERTS AGREED THAT STEEL STIFFENERS HAD PINNED END CONDITIONS

466 The respective experts were agreed that the end conditions of the steel stiffeners were all pinned. Wyatt, who had designed the steel stiffeners, had himself acknowledged this to be the case. Wyatt's intention was always for the bolt connection detail of the steel stiffener to the r.c. column to be "pin connections"; in other words, there was no moment restraint, both vertically and horizontally, and that was the way that all of the design work was done, *i.e*. with no moment transfer. Wyatt was also of the view that the connection of the vertical steel stiffeners to the r.c. floor slab above (*e.g*. section E-E on page 57 of Gary Ng's AEIC) would behave as a pinned connection. What this translated into in terms of the lateral deflections of the steel stiffeners under lateral load was that one would see much greater lateral deflections of the steel stiffeners under the same lateral load as compared to the r.c. stiffeners.

467 As Jones explained it, the vertical steel stiffener would deflect by a significant magnitude under lateral load. However, instead of holding the vertical steel stiffener back and mitigating the amount of deflection experienced by the same, the horizontal steel stiffeners would tend to move out themselves - since they were connected to the vertical steel stiffener by a pinned connection, they would tend to follow the movement of the vertical steel stiffener as the latter deflected laterally.

468 In other words, for the steel stiffeners, when the vertical steel stiffener deflected by 156 mm under lateral load (at the intersection of the vertical and horizontal steel stiffeners), the horizontal steel stiffeners would be pulled along with the vertical steel stiffener (at the end connecting to the vertical steel stiffener) by 156 mm – this was instead of the vertical and horizontal steel stiffeners acting together to "stiffen" the brick wall.

469 Clearly, in this regard, the steel stiffeners were not equivalent to the r.c. stiffeners either, if they did not even serve the purpose of "stiffening" the brick walls (unlike the r.c. stiffeners).

(II) DISPUTE AS TO END CONDITIONS OF R.C. STIFFENERS

470 The experts differed as to the end conditions of the r.c. stiffeners in the calculation of deflection under lateral load.

471 Dr Ting's assumptions in this regard were that the end conditions of the r.c. stiffeners were all fixed, such that all the ends of the vertical and horizontal r.c. stiffeners were fixed, as well as at the intersection of the vertical and horizontal r.c. stiffeners.

472 On the other hand, Jones considered that for analysing the deflection of the r.c. stiffeners under lateral load, the following assumptions were appropriate:

(a) the vertical r.c. stiffener would have a fixed end condition at its bottom end, and a pinned end condition at its top end (due to the construction process);

(b) the horizontal r.c. stiffeners would have pinned end conditions at both the ends connected to the r.c. column, as well as at both the ends connected to the vertical r.c. stiffener (see also the Table of Results at page 3 of D-29).

473 As such, Jones' assumptions as to the end conditions of the r.c. stiffeners under lateral load may be summarised as follows: all the end conditions of the r.c. stiffeners, save for the bottom end of the vertical r.c. stiffener, were pinned.

474 Jones said in his 1st AEIC that:

> The reinforced concrete horizontal tie beams are fixed **monolithically** to the reinforced concrete columns…
>
> The JTC design calls for reinforced concrete vertical and horizontal stiffeners to support the brickwork walls. The JTC stiffener details do not indicate the connection or interface details with the horizontal and vertical reinforced concrete structure. Where constructed, the **conforming detail adopted and approved on site by JTC is a monolithic connection** without any movement capability…
>
> (Emphasis added)

475 The basis for Jones' saying that the vertical r.c. stiffener having a fixed end condition only at the bottom end was as set out at page 11 of D-29, as follows:

> In reality the top of the R.C. member **would not be able to generate continuity due to an absence of grouting after pouring the concrete for the vertical stiffener**. The beam would behave as a propped cantilever. The bottom moment would be limited by the ultimate capacity of the cross section.
>
> (Emphasis added)

476 In respect of the horizontal r.c. stiffeners, Jones also set out his basis for considering the end conditions of the same as pinned, at page 13 of D-29, as follows.

> Horizontal to vertical stiffener / column continuity cannot be achieved due to reinforcement clashing. **End condition therefore approximates to be pinned**.
>
> Note that reinforcement congestion at primary column to horizontal stiffener connection means that **full end fixity cannot be achieved**.
>
> (Emphasis added)

477 However, Dr Ting did not agree with Jones' assumptions of the end conditions. After careful consideration of the evidence of both experts, I preferred the analysis of Dr Ting over that of Jones. In my view, Jones' assumptions were a little extreme and had affected Jones' calculations of the deflections and capacities of the r.c. stiffeners. Although in reality full end fixity cannot be achieved,

however I believed that substantial end fixity could have been achieved if a proper r.c. stiffener design had been executed by a professional engineer and the r.c. stiffener system had been properly built in accordance with the design.

(III) JONES DISAGREED THAT R.C. END CONDITIONS WOULD BE FIXED-FIXED

478 As stated above, had there been a fully-developed r.c. stiffener design, substantial fixed end conditions would have been achieved as intended (whether in respect of bending under lateral load or vertical load). Although Jones asserted that contractors would not have achieved fixity, however in my view that was not necessarily the case. A high degree of fixity, though not total fixity, could well be readily achieved for the connections between the horizontal and vertical r.c. stiffeners, which would approximate a monolithic structure. It would certainly have been JTC's intention for the end connections to be designed and constructed as fixed ends. Indeed, Dr Ting's evidence was that, if the design had called for fixed end conditions, the contractor would have to ensure that the ends were fixed. Accordingly, the lateral deflections would be much reduced due to the greater rigidity of the fixed end connections for the r.c. design when compared to the pin end connections between the horizontal and vertical steel stiffeners for the steel stiffener system.

479 However, since there was no fully-developed r.c. stiffener design for our purposes (because of the change to steel), the parties had considered the comparison of r.c. versus steel on the basis of certain sketches of cross-sections of r.c. members (which Gary Ng, who provided the sketches, said were not intended to serve as the requisite stiffener system at 3.5m x 5m spacings). In respect of these sketches, Jones said that the end connections could not be all fixed, and therefore he had carried out his calculations on the basis that there would be pinned end conditions for some ends of the r.c. stiffeners, and fixed end conditions for the rest.

480 Dr Ting, on the other hand, analysed all the end conditions as fixed.

481 I noted that Wyatt had in cross-examination agreed that at the bottom of the vertical r.c. stiffener, the end condition would be either fixed or closer to a fixed end, while at the top, he did not think it was unreasonable to say that there would be some fixity.

(IV) SKETCH OF R.C. STIFFENERS WERE NOT ONLY BUILDABLE, BUT ALSO CODE-COMPLIANT

482 In this regard, Jones' opinion was premised on the hypothetical construction process which would have been undertaken by the contractor; he did not contend that it was impossible to construct the end connections of the r.c. stiffeners as fixed.

483 In cross-examination, Jones was asked whether, on his comments on fixity of the r.c. end connections, his position was that fixity in the way that Dr Ting had described was physically impossible or that it would not be done in practice. Jones' answer was that it would not or should not be done in practice, because that (if done) would be dependent upon non-compliance with the code in terms of the cover to the rebars, which related to durability and fire protection.

484 When asked further about this, Jones confirmed that the sketches of the r.c. stiffeners were indeed "buildable", and that his view was simply that they were impractical, and would not be done in practice.

MR MANIAM:	Mr Jones, I hear your reservations about code compliance and durability. Would you accept Dr Ting's view that the sections are in effect buildable? In other words, you can bend the bars to achieve the construct as described by him?
A.	**<u>In terms of buildable, I would have to agree, yes</u>**, but they are impractical and would take a significant effort on the part of a contractor to achieve it in practice.

(Emphasis added)

485 However, as was apparent from P-10 (MB1 stiffener) and P-18, the reality was that JTC intended fixed end conditions, and that could and would be done in practice.

486 Dr Ting has also prepared sketches of the rebar detail at the connection of the vertical and horizontal r.c. stiffeners (see P-21), and had given evidence that not only would it be "buildable", it would be quite simple for the contractor to build it, and there would therefore be no clashing of rebars at the connection.

487 This was even though it had previously been suggested to JTC's factual witness, Gary Ng, that the sketches of the r.c. stiffeners showed that that design was "not buildable". Gary had however clarified that the 4T16 details on both pages 31 and 32 of his AEIC had been built before. Further, specifically in respect of the horizontal r.c. stiffener, Gary Ng disagreed that the original detail of 125 x 450 (4T16) was not buildable, and his evidence was that there were JTC projects in which that detail had been used.

488 Dr Ting's evidence in this regard was that the sketches of the r.c. stiffeners would not only be buildable, but would also comply with the code.

489 Dr Ting referred to BS8110-1 (at D-9), under section 3.3.3, at Table 3.3, for his position that, where exposure was mild (as in the present case), a nominal cover of 15 mm to the rebars in the r.c. stiffeners was allowable if Grade 35 concrete was used.

490 Jones, on the other hand, said that the minimum nominal cover to the rebars in the r.c. stiffeners ought to be 20 mm, while for the r.c. column, the minimum nominal cover ought to be 40 mm.

491 Even though Jones was referred to an extract from "*Examples of the Design of Reinforced Concrete Buildings to BS 8110*" where Table 3.3 was explained, he refused to agree with Dr Ting that the nominal cover to the rebars required for compliance with the code was 15 mm instead of 20 mm. This was on the basis that that situation was only allowed where a systematic checking regime was established to ensure compliance with limits on free-water/cement ratio and cement content, and he did "*not believe the Singapore construction industry is ready for such systematic checking regimes*". Jones was also of the view that Dr Ting's opinion was dependent upon using a maximum aggregate size of 15 mm, which although possible, was "*most unusual and unreasonable to expect a contractor to undertake this, unless specific provisions have been put in the specification and contract to allow it*".

492 Jones' reluctance to agree with Dr Ting on the nominal cover was difficult to appreciate when

considering the nominal cover to the rebars in the (much larger) r.c. columns.

493 Dr Ting's interpretation of the nominal 40 mm cover in relation to the r.c. columns was that the cover required referred to the column rebars, rather than the rebars in the r.c. stiffeners which might pass into the r.c. column. Jones disagreed with Dr Ting on that. His opinion was that one could not reduce the cover locally to add in reinforcement from the stiffeners.

494 However, as the Court noted, if that was done, it would result in a very small and localised reduction in the cover, over that rebar from the stiffener, and the whole column would still have got a lot of cover leftover. Jones, however, still found this unacceptable.

> COURT: But you see, the cover is -- 40 mm is over the whole big vertical column, and here we just need a little bit and then the metal bar can go in, so the only effect is a very localised reduction in the cover, just by about the diameter of that dowel bar.
>
> A. Yes, your Honour.
>
> COURT: You mean that is terrible -- I mean you just can't accept that?
>
> A. I wouldn't accept it, no.
>
> COURT: You have still got a whole lot of -- you know -- cover left for the whole beam which is huge, you know?
>
> A. **<u>Yes</u>**, your Honour.
>
> COURT: (unclear) one small little bit at where the beams are, that's all, you mean it's very big deal?
>
> A. Durability is a big deal, your Honour. The concrete cover is there to protect against carbonation. When carbonation extends from the outer face of the column to reinforcement, there is no effective cover left, it means that you then can get corrosion of the reinforcement, so what will happen in practice is that the reinforcement from the stiffener in the column cover will start to corrode first -- and **<u>we're talking many years down the line here, by the way, not immediately</u>**.
>
> Immediately there's not an issue, you've got to look further down.
>
> (Emphasis added)

495 Accordingly, JTC submitted that Jones' position was extreme and unjustified, and that Dr Ting was correct when he considered that the sketches of the r.c. stiffeners were not only buildable, but also code-compliant. Thus, although Jones said that the inability for the end connections of r.c. stiffeners to be fixed would affect cover and cracking which would affect the resistance of the r.c. stiffeners to bending, this was indefensible in light of the above. Counsel for JTC rightly contended that it was obvious that SEC and Jones were driven to take extreme positions to attack the r.c.

stiffeners, so that the steel stiffeners did not suffer in comparison. In any event, were there to be no change from r.c. to steel stiffeners, it would be likely, in my judgment, that the PE tasked to undertake the design would have to fully develop and design a finished r.c. stiffener system that would not only have to work adequately as a stiffener system and be fit for its purpose but also be "buildable".

496 I was driven to conclude that had Jones carried out his analysis of the lateral deflection of r.c. stiffeners based on (a) end conditions that were closer to those of fixed end conditions rather than pin end conditions, or (b) partially fixed end conditions, he would have arrived at much smaller values for those deflections, and he would certainly have found that the r.c. stiffeners would outperform the steel stiffeners. In my view, all the end conditions for the fully developed r.c. stiffeners would more likely be substantially fixed rather than fully fixed or pinned.

b. R.c. would perform better than steel under compression/axial load (packing case)

497 As discussed above, Jones said that the compressive strength of the brickwork was comparable to that of the horizontal steel stiffeners in a packing scenario.

498 In Table 5 of D-29, Jones calculated the compressive strengths of the different materials in the brick wall set-up, as follows:

(a) horizontal r.c. stiffener - compressive strength of 1050 kN/m;

(b) horizontal steel stiffener (H-150) - compressive strength of 101 kN/m;

(c) horizontal steel stiffener (H-250) - compressive strength of 46 kN/m;

(d) horizontal steel stiffener (H-300) - compressive strength of 61 kN/m;

(e) brickwork (per m length) - compressive strength of 52.6 kN/m.

499 Thus, it was apparent from Table 5 at page 22 of D-29 that the horizontal r.c. stiffener of 125 x 450 dimensions had the highest compressive strength by far - of more than ten times the compressive strength of the horizontal steel stiffener (H-150), with the next highest compressive strength calculated by Jones of 101 kN/m for the H-150.

500 Dr Ting has also tabulated the respective compressive strengths of the r.c. stiffener, steel stiffeners, and brickwork, in his exhibit P-23. Dr Ting's own values of the compressive strengths of the various materials in the set-up were as follows:

(a) horizontal r.c. stiffener - compressive strength of 1,584 kN/m;

(b) horizontal steel stiffener (H-150) - compressive strength of 149 kN/m;

(c) horizontal steel stiffener (H-250) - compressive strength of 32 kN/m;

(d) horizontal steel stiffener (H-300) - compressive strength of 43 kN/m;

(e) brickwork (per m length) - compressive strength of 150 kN/m.

Dr Ting also said in cross-examination that the compressive strength of brickwork was about 415 kN/m.

501 As such, on Dr Ting's evidence as well, the compressive strengths of the horizontal steel stiffeners of H1-250 and H1-300 paled significantly in comparison with the other materials in the brick wall set-up, being some three to four times weaker than the brickwork (not to mention r.c. stiffeners) in the set-up.

502 Wyatt also agreed that if one was to look at pure compression, the r.c. stiffener would have a higher strength than the steel stiffeners.

503 It was therefore not surprising that Sim, the key factual witness from SEC, also agreed that r.c. stiffeners would be better than steel stiffeners under compression.

COURT:	You are telling me that an RC lintel of the same dimensions or, rather, whatever the dimensions of the RC lintel were supposed to be, would perform in the same way as your steel lintel, or your steel lintel will perform in the same way as the RC lintel?
	…
	I don't know the basis of the comparison they used but let's assume it's the same size. They may say the concrete lintel is of a different size. Let's start with the same size first. All the dimensions are the same. Let's say the concrete lintel may have a few internal stiffeners -- I don't know what they are. Let's assume it's a normal concrete lintel, the same size as this metal lintel which is hollow and welded. **<u>Load for load, and I keep loading, which one will fail first</u>**? Whether the concrete fails by crushing or what, I don't care. Which will fail first, in your honest view?
A.	The steel lintel will --
COURT:	Fail first. Therefore, in terms of safety factor to carry any load, which one has the greater safety factor?
A.	See, it all depends. Like I say, it depends on the design, how much --
COURT:	The same size; which will carry more load?
	…
A.	That's what I was saying, that if you are saying you don't care about this, just put them all together, same size, same --
COURT:	And apply a compressive load.
A.	Normally, generally, concrete have a more -- it's better in compressive, steel is not so good, so that is one of the basis.

(Emphasis added)

504 Sim also agreed that "equivalent" meant that the steel stiffener had to be of the equivalent strength as the r.c. stiffener.

> COURT: When you said design the equivalent lintel or I give you the lintel size, there's no question of the load of the wall any more. **<u>You take the strength of that element and find something which is an equivalent strength; am I correct? Is that the meaning of "equivalents"?</u>**
>
> A. **<u>You are corre</u>**ct --
>
> COURT: If that thing I give to you can take 100 times the weight of the wall, you don't care about the weight of the wall; you design something which can take exactly 100 times that, the height of the wall, because of my safety factor. Correct?
>
> A. Yes, you're correct.
>
> …
>
> COURT: Yes, I'm asking you as an engineer -- I'm asking you as a civil engineer?
>
> A. As an engineer, we would also look at it, if there's a requirement to build, what is the requirement to build for -- what is the beam supposed to support.
>
> COURT: What is the beam supposed to support we know, but when I ask you to design something equivalent to the strength of my beam and I give you besides my beam -- let's assume another question**<u>. Let's assume I don't tell you what I'm using it for. I say this is a beam. Design something equivalent in steel. What would the engineer do? You tell me</u>**.
>
> A. He would do exactly what your Honour said. That's a hypothetical question that he doesn't know.
>
> COURT: Right. If I tell you to support the weight of that wall, do I need to tell you the beam -- the beam you just have to design, take the weight of the wall and design, you don't have to do reverse engineering. Would I be correct?
>
> A. Correct.
>
> (Emphasis added)

505 Given that the witnesses for both JTC and SEC were agreed that the compressive strengths of the horizontal steel stiffeners were much lower than that of the r.c. stiffeners, it was a foregone conclusion that the r.c. stiffeners would have been better than the steel stiffeners under compression.

506 This was in addition to SEC's own expert's opinion that the steel stiffeners were the "***weakest link***" under compression in the packing scenario. Further, as evident from the relatively high compressive strength of the r.c. stiffeners when compared to the steel stiffeners, if Jones' theory of the defects being due to high compressive stresses were to be correct, then the r.c. stiffeners would easily have withstood such stresses/forces, whereas the steel stiffeners would and did not.

c. R.c. better than steel in the characteristics set out in Tables 1 to 5 of D-29

1. TABLES 1 AND 2, PAGE 21, D-29

507 Jones calculated the bending stiffness of r.c. and steel stiffeners, using uncracked sections (Table 1) and cracked sections (Table 2) of the r.c. stiffeners.

508 The bending stiffness of the r.c. stiffeners as compared to the steel stiffeners was greater, and the steel stiffeners were not better than or equal to the r.c. stiffeners in this regard.

(I) DISCUSSION ON CRACKING

509 Dr Ting gave evidence that, in P-38, he carried out his calculations for the cracked r.c. stiffeners based on the "realistic" crack length along the span of the stiffener. Dr Ting's approach was basically to locate the regions along the r.c. stiffener where the applied moment exceeded the cracking capacity, and then assume that the r.c. stiffener was cracked at those areas. Dr Ting had thus indicated in P-38 the cracked region and the extent of the cracked region.

510 As such, Dr Ting had shown in the rows third and second from the bottom of page 1 of P-38, the length of the cracked section at the top and bottom of the vertical r.c. stiffener respectively.

(II) UNDER LATERAL LOAD

511 Firstly, Jones himself admitted that when considering the bending stiffness of the stiffeners under lateral load, the right-most columns of Tables 1 and 2 were actually not applicable, as E_{long} was only applicable to long term loading; since the lateral load applicable to the site was wind load and not a long-term load, therefore E_{short} should be used.

512 Even when considering the bending stiffness of the stiffeners under wind load using E_{short}, for the uncracked sections, the bending stiffness of the r.c. stiffeners was greater than the bending stiffness for the steel stiffeners (for all types of stiffeners) .

513 More specifically, the bending stiffness for the horizontal r.c. stiffener (of 125 x 450 dimension used by Jones) was calculated to be 1.98E+12 Nmm^2, as compared to 396.1+09 Nmm^2 (for H1-150), 395.5E+09 Nmm^2 (for H1-250) and 556.2E+09 Nmm^2 (for H1-300) – Table 1, page 21, D-29.

514 Similarly, the vertical r.c. stiffener (of 125 x 300 dimension used by Jones) was calculated to have a bending stiffness of 1.32E+12 Nmm^2, as compared to 478.1E+09 Nmm^2 for the V1-200 vertical steel stiffener.

515 Even for the cracked section analysis (see Table 2, page 21, D-29), although the values of the bending stiffness for both vertical and horizontal r.c. and steel stiffeners were close, Jones agreed that if the r.c. stiffener was uncracked or was cracked to a lesser extent than shown in the cracked section analysis, one would get a higher bending stiffness value as compared to those stated at

Table 2.

516 Jones also agreed that if the assumption of fixity was one of either full fixity or even partial fixity (which was not what Jones assumed) and although the bending stiffness remained the same, the deflection experienced by the stiffeners would be reduced.

Q. If your reinforced concrete end connections achieved full fixity or even partial fixity, it would increase the bending stiffness of the members as compared to the analysis here?

A. No, the bending stiffness remains the same.

Q. What, then, would be the effect of fixity at the ends?

A. It reduces the span moment. The applied no moment along the span is reduced.

Q. The net result is that you would reduce the amount of deflection produced?

A. Correct.

517 Dr Ting also considered that the r.c. stiffeners would have fixed end conditions at all connections, under lateral load; Jones however disagreed with Dr Ting in this regard, and considered that only the bottom of the vertical r.c. stiffener and at the intersection of the vertical and horizontal r.c. stiffeners would have fixed end connections.

518 JTC contended that, on Jones' own uncracked section analysis (as set out at table 1, page 21, D-29), the r.c. stiffeners would have greater bending stiffness and deflect less under lateral load, as compared to steel stiffeners. JTC further contended that if Jones' overly stringent assumptions of the cracked lengths of the r.c. stiffeners, as well as the assumptions in relation to fixity, were moderated (and I believed that would have been more akin to the on site conditions), the bending stiffness of the r.c. stiffeners as compared to the steel stiffeners would still be greater in a realistic cracked situation. I found no good reason to disagree with these contentions of JTC.

(III) UNDER VERTICAL LOAD

519 First, it must be emphasised that Dr Ting did not agree with Jones that E_{long} should be used in analysing the bending stiffness of the stiffeners, and considered that we should look at the E_{short} column, but not the E_{long} column in this regard.

520 In any event, Table 1, page 1, of D-29 showed that the uncracked r.c. stiffeners had greater bending stiffness than the steel stiffeners (for both vertical and horizontal stiffeners) under vertical loads. This was borne out by the calculations, whether on the bases of using E_{short} or E_{long} (although Dr Ting did not agree with using E_{long}). As such, for uncracked sections, r.c. stiffeners were better than steel stiffeners in terms of bending stiffness, even on Jones' own evidence.

521 For the cracked sections however, the values of the bending stiffness for both vertical and horizontal r.c. and steel stiffeners were close.

522 I therefore accepted JTC's submissions that for the uncracked sections, the r.c. stiffeners were clearly better than the steel stiffeners under vertical load. For the cracked sections, if Jones' assumptions were moderated, the bending stiffness of the r.c. stiffeners as compared to the steel stiffeners would still be greater.

2. TABLE 3, PAGE 22, D-29

523 In terms of axial stiffness, JTC contended that the vertical r.c. stiffener was better than the vertical steel stiffener.

524 Here, Jones calculated the axial stiffness (using E_{short}) of the vertical r.c. stiffener (of dimension 125 x 200) to be 6.75E+08 N, while the axial stiffness of the vertical steel stiffener (V1-200) was found to be 3.91E+08 N. As such, it became clear that, using E_{short}, the vertical r.c. stiffener had a greater axial stiffness than the vertical steel stiffener.

525 When E_{long} was used, the axial stiffness calculated by Jones for the vertical r.c. stiffener and the vertical steel stiffener were 3.38E+08 N and 3.91E+08 N respectively.

526 However, as the axial stiffness of the member was calculated by taking the E-value multiplied by the cross-sectional area of the member, the axial stiffness of the vertical r.c. stiffener would therefore increase by 50% if Jones had carried out the calculations using the 125 x 300 vertical r.c. stiffener (which was consistent with JTC's position, and which dimensions Jones had used in Tables 1 and 2, page 21 of D-29).

> Q. Mr Jones, if you can look at table 3, which is over the page on page 22, that is a calculation of axial stiffness. I see that the dimensions you have used for the vertical reinforced concrete stiffener are 125 by 200, rather than 125 by 300, which was used in tables 1 and 2. Can you tell us why there is a distinction there?
>
> A. That arises from my original analysis, where the detail was 125 by 200 by 4T16s. I didn't redo that analysis for the later detail I was provided with, which showed a 300 by 125 by 2T25 section.
>
> Q. If we were to rework this for a 125 by 300 section, my understanding is that that produces a value that is 50 per cent higher in terms of axial stiffness, because the calculation is based purely on the cross-sectional area. Would that be correct?
>
> A. That is correct.

527 As such, I accepted that for the 125 x 300 dimension of vertical r.c. stiffener (which dimensions JTC said that the sketch by Gary Ng was in respect of), the axial stiffness of the vertical r.c. stiffener would be greater than that of the vertical steel stiffener, whether E_{short} or E_{long} was used. Accordingly the r.c. stiffeners would be better than steel stiffeners in that respect as well. I noted that even on Jones' own calculations, the steel stiffeners were neither equal to nor better than the r.c. stiffeners.

d. R.c. better than steel as it would not have twisted / side-swayed

528 With regards to resistance to twisting and side-sway, I also found that the r.c. stiffeners would clearly have been better than steel stiffeners. The r.c. stiffeners would not have twisted or deformed under compressive and lateral loads in the first place, unlike the steel stiffeners which in fact had twisted and/or side-swayed.

529 Wyatt's own rectification proposals attested to the superiority of the r.c. stiffeners to the steel stiffeners in this regard. The rectification proposals had suggested that the webs be bolted together, and the steel stiffeners be filled with grout to strengthen them. Wyatt himself said that the idea was to create something that was akin to an r.c. stiffener, with the bolts acting like rebars, while the grout in the stiffeners worked as the concrete.

MR MANIAM:	Your Honour, for rectification purposes, the bolts would actually go through the webs, and they would actually connect the two webs. I think that would be correct, Mr Wyatt?
A.	My understanding is that JTC have adopted similar details to what we had previously detailed for some of the earlier rectifications, and the purpose of the bolts is to try and -- once the inside of the stiffener is grouted, the purpose of the bolts is to actually try and make the section act as a composite member. In other words, there's a -- to improve the bond between the steel and the infill concrete.
COURT:	The infill concrete is in the stub itself? The beam and the stub --
A.	It's within the full length of the stiffener.
COURT:	The beam and the stub is integral? The concrete goes all the way through?
A.	That's correct.
COURT:	And then the concrete goes around the bolt also?
A.	That's correct.
COURT:	The bolt cannot be removed at all?
A.	Correct. And the idea is to create something that's a little bit like a reinforced concrete member, with the reinforcement on the outside.

e. R.c. structure would attract less load under creep and shrinkage

530 It was a trite proposition that steel would expand and contract significantly due to thermal changes, while the r.c. frame would only creep and shrink; as such, in a brick wall set-up with steel stiffeners, the steel stiffeners would tend to move in the **contrary direction** from the r.c. frame. Whilst the r.c. frame would creep and shrink, the steel stiffeners would expand and contract.

531 On the other hand, in the hypothetical set-up with r.c. stiffeners, the r.c. stiffeners would creep and shrink **in tandem** with the r.c. frame. The r.c. stiffener itself would therefore not suffer the same amount of stress as the steel stiffener.

532 In that event, the r.c. stiffeners would also be better than the steel stiffeners, as the damage (if any at all) would be caused to the brickwork/plaster rather than to the stiffeners themselves, unlike in the situation with the steel stiffeners, where the damage would be caused directly to the steel stiffeners themselves as well as to the brickwork/plaster.

iii. What if r.c.?

a. R.c. would deflect less than steel under lateral load

533 If r.c. stiffeners had been installed in the Development instead of steel stiffeners, the most obvious difference would have been that the deflections of the r.c. stiffeners under lateral loads would have been much smaller than what was calculated for the steel stiffeners (as would be apparent from the calculations carried out by the respective experts – see P-38 and D-29 at page 3).

534 Further, not only would the deflection of the r.c. stiffeners under lateral loads be smaller than the deflection for steel stiffeners, the deflection of the r.c. stiffeners would also have been within the code limits for deflection.

535 All in all, what this meant was that the plaster / render would not have been damaged or cracked if r.c. stiffeners were installed instead of the steel stiffeners. The r.c. stiffeners would not have twisted or deformed under compressive and/or lateral loads in the first place, unlike the steel stiffeners which I found had in fact twisted, side-swayed or buckled. Indeed, Wyatt himself had earlier prepared rectification proposals which suggested that the webs of the steel stiffeners be bolted together, and the steel stiffeners be filled with grout to strengthen them. The result of all this was to create something equivalent to an r.c. stiffener. Wyatt thus indirectly acknowledged through his rectification proposal to JTC that use of r.c. stiffeners, instead of steel stiffeners, would have been better.

b. R.c. would be fine under compression

536 By Jones' movement theory, which relied on the compressive loads acting on the set-up as the explanation for the defects observed on site, the r.c. stiffeners would have been fine under the compressive loads, and JTC would not have had to repair the r.c. stiffeners like they had to do for the steel stiffeners.

537 In this regard, SEC's witnesses appeared to be saying that although the r.c. stiffeners would not have suffered deformation like the steel stiffeners, the brick walls would have failed instead since the r.c. stiffeners were stiffer. SEC submitted that Jones had clearly explained that the phrase "*weakest link*" was only in relation to the fact that the steel stiffeners, being the most compressible element in the brick wall would tend to give way before the brick walls. In other words, the steel stiffener was the "*weakest link*" in the packing case.

538 As such, Wyatt had said that the disadvantage of fixed end (*i.e.* with the r.c. stiffeners) was that "*all the anchors have to work a lot harder*" and Wyatt believed that "*it's undesirable to have failure of the anchors, because if the anchors fail, the whole system fails*". Further, Wyatt believed that if r.c. stiffeners had been used instead of steel stiffeners, the brick walls would have failed suddenly, under compression.

539 On the other hand, with the steel stiffeners installed in the brick walls being the most "compressible" material in the set-up, the damage would not have been caused suddenly, but would have been apparent in the compressing steel stiffeners. As such, one might even be able to read into Wyatt's evidence as set out in the preceding paragraph, that having steel stiffeners would be safer than having r.c. stiffeners as the brick walls would not fail suddenly in a set-up with steel stiffeners.

540 To this, JTC responded that the damage which SEC's witnesses say would result was exaggerated.

541 I agreed with JTC's submissions that this was an unmeritorious argument, for the following reasons:

(a) if JTC had wanted stiffeners that would fail in order to indicate stress in the brick walls, JTC would have contracted for that - however, what JTC had in fact contracted for was a functional stiffener system;

(b) according to Jones' movement theory, the vertical steel stiffeners (as the weakest link in the whole set-up) would have no means of escaping the creep and shrinkage of the r.c. frame. The vertical steel stiffeners would not creep and shrink with the r.c. frame (as would r.c. stiffeners, thus alleviating stress); on the contrary, thermal expansion of the steel stiffeners would create even more stress;

(c) the r.c. stiffeners themselves would have been relatively unaffected by movement as they would creep and shrink along with the r.c. frame.

542 If (as per the original contract requirements) r.c. stiffeners had been used in place of steel stiffeners, even if there was damage caused to the set-up due to creep or shrinkage, the expansion of bricks, lateral loads and so on and so forth, the rectification costs which JTC would have had to incur in respect of repairing the damage would have been lower in any event (see the next section for more explanation on this).

c. With r.c., rectification works would have been simpler and cheaper

543 I accepted the submission of JTC that if r.c. stiffeners had been used and defects had arisen, there would not have been a need to replace them. This was unlike the case of the steel stiffeners, an example of which was unit #01-19, where the horizontal steel stiffeners had to be replaced. Some of the drastic rectification works included:

(a) demolishing the brick wall above the horizontal steel stiffeners, and part of the brick wall below the same;

(b) replacing the horizontal steel stiffeners with rolled hollow sections;

(c) jacking back the vertical steel stiffener to its original position and tying it back to the r.c. frame.

544 Even for the brick walls with steel stiffeners which did not require such drastic rectification works, the steel stiffeners would still have to have their webs bolted together, and then grouted and strengthened. This manner of rectification works would obviously cost substantially more than merely replacing the top course of bricks which might have crushed, and re-plastering the brick wall, were the r.c. stiffeners to be used as originally specified by JTC for the Development.

545 Thus, JTC's contention, which I accepted, was that if the stiffeners were r.c. (as per the original design), the rectification works (if any was required at all) would not involve such expense as for the steel stiffeners. The parties would not be confronted with the very substantial rectification costs incurred by JTC, as in the present case.

546 When asked by the Court about the situation (with r.c. stiffeners) where the mortar at the top of the brickwork failed and the render on the outside of the brickwork had popped out, Jones himself agreed that the rectification works would essentially involve scraping off the plaster and re-plastering the brickwork, before repainting the same and then getting the tenants to move back into the units. The repairs would last for another six years and then JTC could then go back and just do it again - it would be cheap and easily done. Furthermore, each time less repair would be required as the creep effect would be less.

COURT: So you just scrap out the whole thing, the whole length and then you replaster? That is cheap, isn't it, for repair work?

A. Once a defect like that has occurred, your Honour, I would tend to put a groove line in, if there wasn't one there already.

COURT: Yes. If they repair and put a groove line, it is better. That is a more expensive repair, but if they do a quick and dirty job, then they just scrap it off and repaint and then get the tenants to move in.

A. **<u>That is possible</u>**, your Honour.

COURT: And it lasts another six years.

A. Yes. Then you go back --

COURT: No problem.

A. -- **<u>do it again</u>**.

COURT: Yes, do it again, every six years, part of the tax relief for expenses for the company.

A. Right.

COURT: It wouldn't be that massive a job.

A. No, the only big part of it is the scaffolding.

(Emphasis added)

547 This was consistent with Jones' evidence when cross-examined earlier on fair repair methods, when he agreed that one method of repair which could be used (had r.c. stiffeners been used and if indeed the top course of bricks in the brick walls was crushed) would be to replace the top course of bricks, creating a compressible joint at the top (if one was concerned about further movement), and then replastering the brickwork. However, Jones went even further to offer his view that, it was likely that quite a bit of movement would have already taken place by that time, and therefore it could well be that horizontal compressible joints would not even be needed.

548 However, when Jones was informed that JTC's position was that the rectification costs incurred in respect of steel stiffeners as compared to what it would have cost in respect of r.c. stiffeners was substantially more, Jones did not expressly agree with the same, despite agreeing earlier that the rectification works in an r.c. situation would be simple (see preceding paragraph). Instead Jones tried to say that the rectification procedure could be the same, and that in the steel stiffener situation, JTC might possibly "*have to do one or two additional repairs if distortion is significant*", although Jones considered that, "*from the photographs, the actual distortion that you can see is not really that significant*". Jones also said that he was not sure that it was necessary to grout the steel stiffeners once the load was relieved.

549 Once again I agreed with JTC's submissions that JTC would have been better off had SEC not proposed a substitution of steel for the r.c. stiffeners originally contracted for, especially since the steel stiffeners were not as good as or better than the r.c. stiffeners, and eventually resulted in JTC expending more costs to rectify the same.

d. Defects in question would not have arisen if r.c. stiffeners were not substituted with steel stiffeners

550 In any event, as discussed above, JTC's position was that had r.c. stiffeners been installed instead of steel stiffeners, the deflections in question would not have arisen as tolerance in compression would have been provided by the following:

(a) Crushing of the mortar (soft joints) between the bricks;

(b) Air voids in the mortar between the bricks; and

(c) Vertically-placed or flipped bricks.

551 Further, even if there would have been defects in a set-up with r.c. stiffeners (which JTC denied), the defects would not be the same as the defects in question in the present case. As stated in the preceding section, if there was indeed damage caused in a set-up with r.c. stiffeners, JTC

would at most have had to replace some bricks and replaster the brickwork.

552 As such, JTC rightly submitted that it would be an exaggeration to say that the same defects would be observed and same repairs would have been carried out, had the stiffeners used been r.c. instead of steel.

iv. Allegation of defects at locations even with r.c. stiffeners

553 SEC raised various arguments as to why the defects in respect of which JTC was claiming against SEC were not caused by and/or attributable to the steel stiffeners. One of these arguments was the allegation that, even in areas where r.c. stiffeners were installed, SEC observed instances where the brick walls at such areas of the Development had bulged or cracked (see paragraph 17(d) of the Defence).

554 In this regard, in response to JTC's requests for particulars of the precise locations which SEC was alluding to, SEC's answer was:

> The external façade of SS1000 units #01-01, #03-01, #05-01 and staircase B at the rear of these units...

555 As such, these particulars filed on 21 November 2006 only referred to the external facade of SS1000 units #01-01, #03-01, #05-01 and staircase B at the rear of those units; these allegations had been specifically been addressed in the report of JTC's expert Dr Ting.

556 SEC subsequently applied to Court for leave to file further Further and Better Particulars of the Defence to add more locations where SEC was alleging r.c. stiffeners were found, with bulging and/or cracking brick walls, as follows:

> The external façade of SS2000 units and staircases with reinforced concrete lintels/stiffeners…

557 In this regard, SEC also obtained leave for an AEIC of Ng Chong Boon (an ex-employee of SEC) and further expert reports. JTC disputed Ng Chong Boon's assertion that there were r.c. stiffeners (as distinct from r.c. tie-beams) at the 1st & 2nd, 4th & 5th, 7th & 8th storeys of the SS2000 units.

558 In any event, JTC was not claiming damages in respect of the re-plastering of hairline cracks at r.c. stiffener locations.

559 During the trial, SEC also referred to defects at window frame areas. SEC highlighted the buckling of window frames below r.c. tie beams at the SS 3000 series as an example. These were window panels of 5 windows, consisting of a panel with 2 windows and another panel with 3 windows that were joined together. It was observed however that the buckling occurred at the joints between the 2 window panels where there was a single flat piece. As Dr Ting explained, this flat piece had very little compressive strength. Also, the window frames could themselves be of poor quality.

560 In any event, it would appear that SEC was seeking to argue that if there were some defects at r.c. stiffener locations that must mean that none of the defects at the steel stiffener locations was steel stiffener-related, but that the defects were a sign of some systemic deficiency in the building structure.

a. At SS1000 units

561 From available information, the stiffeners at the external façade of the SS1000 units were steel

stiffeners, and not r.c.

562 In any event, defects at r.c. "stiffener" locations were not reviewed with any seriousness at all in SEC's expert's reports - for example, at paragraph 4.4.2 of Jones' 1st AEIC, he concluded that where the stiffeners had been constructed of r.c., the same behaviour had occurred as no horizontal compressible joint was provided at the underside of the r.c. structure. However, there was no substantiation of this assertion whatsoever. In particular, there was no specific mention of the SS1000 units.

563 Without proper substantiation of this assertion, SEC was clearly just clutching at straws in its attempt to divert attention from the deficient steel stiffeners.

b. At SS2000 units

564 SEC also asserted that there were r.c. stiffeners at SS2000 units. However, apart from the fact that SEC only pleaded this assertion at a very late stage, it was also puzzling why they would want do so, as the r.c. stiffeners which Ng Chong Boon referred to were actually r.c. tie-beams. Being structural in nature, these r.c. tie-beams were of an entirely different species from the hypothetical r.c. stiffeners.

565 Ng Chong Boon's AEIC stated that he was asked to confirm which areas of the side elevations of the SS2000 units contained steel stiffeners and which areas contained r.c. stiffeners, which he said that he was in a position to do because he was based at the site since the commencement of the superstructure works for the SS2000 series.

566 A simple sketch apparently showing the steel stiffeners and r.c. stiffeners was then exhibited at "NCB-1" of his AEIC, purporting to show that steel stiffeners were installed at only the 3rd, 6th and 9th stories of the SS2000 units; the stiffeners at all other stories were apparently r.c.

567 In the 2nd AEIC of McGowan's, filed on 9 July 2007, McGowan also referred to Ng Chong Boon's AEIC and evidence in that regard, in arriving at his conclusion that "*defects occur irrespective of whether there is a steel lintel, reinforced concrete lintel or a reinforced concrete beam present within the wall*".

568 McGowan also provided a table (Table 3.2, at pages 11 to 13) purportedly showing the locations of defects with reference to the elevation sketches prepared by Setsco in their AEIC.

569 Apart from the objectionable inclusion of alleged defects at r.c. lintel / stiffener / beam locations at SS3000 units (which had not been pleaded) and of alleged defects at elevations 1 and 3 of the SS2000 units (Ng Chong Boon's evidence was that only elevations 2 and 4 had r.c. "stiffeners"), this assertion of defects at r.c. stiffener locations at SS2000 units was simply misleading as there were no r.c. stiffeners at SS2000 units.

570 JTC's witness, Mr Tan Eng Hong (the Clerk of Works for rectification works), had given evidence that there were r.c. beams and steel c-channels filled with mortar at the 1st 2nd, 4th, 5th, 7th and 8th stories (*i.e*. where SEC said there were r.c. "stiffeners"). He also confirmed that there were steel stiffeners at the 3rd, 6th, and 9th stories.

571 Even SEC's own witnesses confirmed that the r.c. "stiffeners" which they were referring to were r.c. tie-beams.

572 When shown the as-built drawings of the SS2000 units in cross-examination, Ng Chong Boon himself confirmed that the r.c. "stiffeners" which he was referring to were the ones as shown on the as-built drawings (which were r.c. tie-beams, rather than stiffeners), and Ng Chong Boon himself would call them either "*RC lintel* ***or tie beam***" (emphasis added).

573 McGowan had also confirmed said that the r.c. "stiffeners" alluded to in Jones' 1st AEIC were actually r.c. tie-beams; further, it was McGowan who had referred Jones to the existence of the r.c. "stiffeners" at the Development. McGowan further confirmed that if we were to see references to r.c. stiffeners at the Project in Jones's 1st AEIC, we should understand those references as being references to structural r.c. tie beams.

574 Clearly, there must have been some miscommunication somehow, because even McGowan himself was of the opinion that these were the structural r.c. tie-beams (and not r.c. stiffeners).

575 SEC also sought to suggest, with reference to pages 24 and 25 of D-14 - SEC's Bundle of Photographs, that the debonding of plaster at the r.c. tie beam areas, indicated that the steel stiffeners were not the cause of the defects observed on site.

576 However, as Dr Ting explained, this could be due to the contractor not preparing the base material properly before plastering, resulting in the debonding of the plaster.

577 It was also worth highlighting in this regard that Sim also agreed that one could not simply look at the plaster cracks at other areas to conclude that the steel stiffeners were not the cause of the defects.

c. Plaster cracks at staircases due to differential loading

578 In relation to SEC's allegations of plaster cracks and defects at staircases, the movement cracks along the staircase walls were an indication of differential loading between the staircase structure and the main structure – but they were not stiffener-related.

579 Indeed, Jones clarified (of his own accord) that the r.c. members at the staircase areas were actual r.c. structural beams, and not r.c. stiffeners. Jones also agreed that the top course of bricks under r.c. tie-beams at the staircases in the Development which appeared to be crushed (such as those referred to in the JCPL report (at 12 AB 4344; with photographs at 12 AB 4416) were laid lengthwise, at 90 degrees to the typical courses of bricks below them. Jones also agreed that he had previously said that bricks laid that way would have somewhat lower compressive strength as compared to bricks laid the usual way.

580 In this regard, Jones said in re-examination that the photographic evidence of the infill brick wall and the failure of the upper course of bricks on end was actually a classic brick infill wall failure under extreme compressive load. However, he also said that the value of shrinkage and creep in concrete was actually a function of the stress in the concrete and the load that was imparted upon that concrete to cause that stress. It might be the same, it might be more or it might be less than in the primary columns, and one would have to do some calculations.

581 Jones had also previously said that staircase walls had limited application for load-carrying, and were not really walls of great significance in terms of stability. The wall Jones referred to had no steel stiffeners, and was not the subject of a claim by JTC.

582 Moreover, I agreed with Dr Ting that the loads imposed on the main structure and on the

staircase structure were quite different and cracks had resulted from differential loading. This was not relevant in evaluating the cause of the defects at the steel stiffener areas.

583 I further noted that there was no considered analysis of the cracks at staircase areas in SEC's expert's reports. This fact, JTC rightly submitted, would indicate that SEC's own expert was not prepared to support SEC in respect of its vague allegation regarding defects at staircase areas. In my view, the crushed bricks observed at the staircases were simply localised failures due to the bricks being upturned on their ends.

E. Conclusion

584 In my view the horizontal steel stiffeners failed primarily because the vertical web elements supporting the flanges of the horizontal steel stiffeners, as designed by SEC, were too slender. There was also insufficient internal cross-sectional bulkhead stiffening of the box structure of the steel stiffeners at appropriate intervals along the length of the hollow horizontal steel stiffeners to prevent side-sway (*i.e*. parallel movement of the flanges relative to each other under vertical loads such that the rectangular cross-section would distort into a parallelogram), having regard to the expected lateral and non-uniform wind loads acting on the wall panels (not necessarily of the same height), and the weight of the wall on top of the stiffener and the existence of a wall below the stiffener, which would then collectively impose compressive vertical and non-uniform lateral forces on the webs and flanges. Due to the different heights of the wall panels above and below the horizontal steel stiffeners, even a uniform wind load over the entire wall would give rise to differential lateral forces on the top and bottom flanges of the horizontal steel stiffener, which would then introduce torsional moments on the stiffeners, over and above any torsional moment that would result from any non-eccentricities in the construction of (a) the wall sitting on the top flange and (b) the wall below supporting the bottom flange of the horizontal steel stiffeners. Creep and shrinkage of the structural framework and expansion of the brick wall over time would additionally contribute to the compressive vertical loads on the horizontal steel stiffeners, where these compressive loads arising from the creep, shrinkage and expansion movements were not relieved by the mechanisms alluded to earlier (comprising the crushing of the mortar between the bricks, the inherent air voids in the mortar and the vertically-placed or flipped bricks at the top of the wall).

585 The predominant mode of failure appeared to me to be that of web buckling/web bowing and side-sway of the horizontal steel stiffeners leading to the wall defects found on the site mainly at where the steel stiffeners were located. I found that these defects were caused by the inadequately designed and constructed steel stiffeners which SEC proposed, designed, constructed and installed for JTC (in substitution for the r.c. stiffeners). As the horizontal steel stiffeners were designed by SEC to be entirely hollow throughout its length, there was in fact no internal cross-sectional bulkhead stiffening the relatively thin steel box like structure anywhere along the entire length of the horizontal steel stiffener, thus making the steel stiffeners particularly prone to side-sway and web buckling. The vertical steel stiffeners were not sufficiently rigid in preventing the lateral movement (under lateral loads) of the horizontal steel stiffeners away from the vertical plumb line at the location where the horizontal steel stiffeners were joined to the vertical steel stiffeners (to form supposedly a sturdy steel stiffener framework to stiffen the walls). This in my view further contributed to the wall damage found at the Development. There was also evidence of poor welding and installation work at the site which could have further exacerbated the defects found.

586 In the alternative, should I be wrong in my findings on the probable cause of these alleged defects on the walls at the Development, which had arisen some years later and which had appeared mainly where the deformed steel stiffeners were located, I would conclude that in any case JTC would be entitled to rely on the principle of *res ipsa loquitur*. It was for SEC to justify the design,

construction and fitness for purpose of their steel stiffeners, and that their proposed steel stiffeners were at least equivalent to the r.c. stiffeners originally specified by JTC. On the totality of the evidence, I found that SEC had failed to do so.

587 For the reasons stated above, I found SEC to be liable to JTC for the defects and ordered damages to be assessed by the Registrar with costs reserved to the Registrar. I did not make any declaration as prayed for by JTC. However, I further ordered that the SEC would be liable for any further defects arising thereafter which could be proved to be attributable to the steel stiffener system.