SESOC’S VIEW ON DOUBLE TEE SUPPORTS

John Hare

SUMMARY
Since SESOC published their article regarding the loop bar double tee support detail (also known as the ‘pigtail’), there has been considerable debate as to whether the paper was a reasonable response, or whether SESOC may have over-reacted. This paper presents some of the reasons why SESOC took the action they did. The pigtail detail was accepted into use under a different building code regime to that which exists now. Although it has performed adequately to date, there have been no significant earthquakes to fully test it. SESOC’s conclusion has been that, given there is no rational design or analysis that can justify its use, the industry should adopt alternative details that can be verified by rational analysis. Chartered engineers are bound by their Code of Ethics to do nothing less.

INTRODUCTION
In the September 2008 SESOC Newsletter, an article was published noting the SESOC management committee’s concerns regarding the use of the loop bar support detail for flange supported double tees (this detail is more commonly known as the ‘pigtail’). Subsequently a full report and recommendations have been published, including a comprehensive analysis of the detail.

This has generated a lot of interest and comment. This is unsurprising, considering that the detail has been in use for over thirty years. Obviously there are implications for the future design of double tee support systems, but equally it raises concerns over the performance of existing installations.

SESOC did not take this step lightly, and nor did it seek simply to create conflict. But in taking the steps that it has, it has sought to behave responsibly and practically. In an industry that is self-regulating, it is important that organisations such as SESOC provide leadership to the industry, as they can speak for a wider audience than can individuals or even organisations. While individuals may have been questioning the detail for a longer period than SESOC, it is clear that by taking this action, a faster resolution may be achieved.

The purpose of this paper is to present SESOC’s concerns and outline a possible process by which the industry can move forward with safety and without further conflict. Matters to be addressed include reasonable acceptance criteria for testing, critical design requirements, and better definition of the respective responsibilities of designers and contractors.

BACKGROUND
The loop bar detail has been in use in New Zealand for over thirty years. When originally developed, it was tested (for gravity only), and a design methodology developed, believed to have been based on a shear-friction model.

Subsequently further testing has been done as requested, when engineers or specifiers have requested further validation for the pigtail detail. Most of this testing is now unavailable for review, as the tests may have been done informally, may be subject to client confidentiality, or may have been lost.

In recent years, the use of the pigtail, once exclusive to a single manufacturer, has now become widespread. At one stage, use of the detail was subject to informal control, with the developing manufacturer recommending other details in situations involving, for example, heavy wheel loads, repetitive loading, or long spans. However more recently, the detail has been recommended in all such situations, even when designers have specified alternative details.

In the most extreme case SESOC has observed, a pigtail support was recommended for a 16m span, on a slope, suspended on steel beams flanges.

When this matter was brought to their attention, SESOC were concerned with this, and with the apparent inability of rational analysis or design methods to substantiate the use of the pigtail.

PROFESSIONAL OBLIGATIONS
Self-regulation carries with it certain responsibilities. IPENZ sets clear requirements in its code of ethics, which also governs the

---
1 SESOC Management Committee Member, study group convener
minimum ethical standards for Chartered Engineers, in accordance with the Chartered Professional Engineers Act.

The first clause of Part 3 – Minimum Standards of Acceptable Ethical Behaviour by Members states:

1. Take reasonable steps to safeguard health and safety

A Member must, in the course of his or her engineering activities, take reasonable steps to safeguard the health and safety of people.

This should not be taken lightly. Chartered Engineers need to consider all of the reasonably foreseeable combinations of loads and displacements that their designs will be subjected to and ensure that appropriate detailing is employed.

The question that should be asked is – if a proprietary component is specified, what responsibility has the building designer to verify that it meets the specification? Arguably, when the full range of design actions is not able to be verified by the supplier and there is no verification method available to the designer, the designer is failing to meet the minimum standard expressed above if he or she continues to accept the detail.

Under the New Zealand Building Code, the only approved verification method is NZS 3101:- Part 1:1995, including amendments 1, 2 & 3. Therefore, any solution that is not fully compliant with NZS 3101 is by definition an alternative solution, to be accepted at the discretion of the building designer and the BCA. But building designers need to be aware – as they have responsibility for the performance specification that the floor is designed to, they can be assumed to have liability to their clients for the specification being met.

The debate seems even more clear-cut when it is considered that there are other solutions available to designers. These were presented in the SESOC paper, so will not be repeated here, but it raises a further question – if a designer has relatively low-cost alternatives readily available, why accept a detail that does not meet the standards that the codes explicitly require? Putting aside the ethical issues, designers need to consider their liability for a detail that is inadequate, in light of notifications (such as the SESOC article) that have been given.

Further discussion in regard to the designers’ detailed responsibilities follows below.

POSSIBLE VERIFICATION METHODS

Although described in much more detail in the SESOC paper and appendices, the possible verification methods will be summarised briefly here.

1. By design

The first method for verification of the support system is conventional design according to the established NZ standards, or approved alternatives. The New Zealand Building Code cites both the loading and concrete design standards as the approved verification methods. Although it is noted that NZS3101:2006 has not yet been gazetted, it is reasonable to assume that it will be in the future, and could therefore be held to represent current best practice for concrete design.

In order for the pigtail (or any other detail) to be considered code compliant, it is critical that it comply in all respects, for all possible combinations of loads and effects. It is also necessary to ensure that the design methodology is appropriate for the element being considered.

As the support is clearly in a disturbed region, the most appropriate method for analysis and design is strut-and-tie modelling. It should be noted that this method contains an inherent conservatism, as it assumes that the concrete strength in tension is zero. Therefore all actions are resisted either by steel in tension or concrete in compression.

As the SESOC study group demonstrated, there is no viable mechanism in the pigtail detail as it stands, due to the lack of a suitable stirrup or other tension tie to restrain the struts that must form over the potential crack line from the throat of the unit (refer figure 1 below).

Figure 1: Strut and Tie analysis (from [2])
In addition to this, there are several other deficiencies from a strict code perspective:

- The loop of the pigtail is bent to a tighter radius than allowed by code
- The R20 bar in the centre of the loop is tack welded to the loop
- The main bars of the pigtail (generally R12s or R16s) are plain bars with inadequate development (plain bars must have a hook anchorage by code)
- The stirrups for the double tees are generally inadequately anchored – in some cases they are bent out into the precast flange and/or have no bars tight to the bend, but in no cases are they carried up to engage the compression field (in the composite case).

Use of a shear-friction model is often cited as a verification method for the support, but it is an inappropriate application for this methodology. For shear-friction to be applicable, it is critical to have an uncracked solid concrete element to resist the vertical component of the load, in the compression zone. In this case, there is no such element (see figure 2 below).

![Figure 2: Shear friction](image)

Further, as outlined in the recent SESOC paper², the shear friction concept in the Structural Concrete Standard, NZS 3101; 2006 cannot be used for assessing diagonal tension strength of a beam or other structural member. The shear friction equations predict the magnitude of shear force that can be transmitted across a crack. It tells the designer nothing about the stresses that can be sustained by the concrete adjacent to the crack, hence it does not indicate when this concrete will fail due to the formation of a crack, which leads to diagonal tension failure.

Taking all of the above into account, the pigtail as currently detailed cannot be verified by conventional design methods.

2. By Prototype Testing

Appendix B of AS/NZS1170⁶ presents a methodology for the verification of units by prototype testing. For this to be applicable, there are some critical requirements:

- The units being tested should be drawn from the production run
- A test loading regime must be developed that is representative of all possible loads and effects that a unit may be subject to in service
- The test load for a given situation must take into account the variability of all of the constituent materials and parts of the test unit. In the case of the double tee supports, the coefficient of variation must be at least 40%, as the units are reliant on concrete in shear.
- Every time a parameter is varied, a new series of tests should be run.

Clearly, this form of verification would require a tremendous number of tests to allow for the number of variations in the flooring units and load conditions, a practical impossibility for this sort of detail.

The standard clearly states in Appendix B of this method: “The method is not applicable to the testing of structural models, nor to the establishment of general design criteria or data.”

3. By Research Testing for development of a Rational Analysis Method

The final of the general methods of verification is by research testing aimed at investigating the possible failure methods and hence developing a rational analysis method.

This method requires that specific tests be designed to investigate each of the postulated failure mechanisms in order to determine both the likelihood of the particular failure and the influence of the input loads and effects. Possible failure mechanisms have been described in the SESOC paper² so will not be reproduced in detail here.

The postulated failure mechanisms may not develop, as has been demonstrated by the testing to date. In order to test a particular mechanism,
units may need to be pre-cracked, or have crack inducers introduced to the specimen, to achieve the required output.

In parallel with the tests, finite element or other numerical analysis may be used to compare the observed and predicted behaviour. This allows the subsequent development of a design method that can allow for variations in the load conditions and material strengths of the units.

This form of analysis is commonly carried out in universities and other research institutions.

4. Discussion on testing to date

Much of the testing that has been done to date is, with hindsight, of little use now. Although not all of the previous tests have been reviewed (or available for review), relatively few have adequate records and/or the test specimens and test set-up are not suitable. So it is not practically possible to back-analysis these tests to good effect.

Until recent tests undertaken by BRANZ, no units had been subject to any review of possible performance under seismic actions, and full material properties had not been recorded. Even in the most recent testing, no pigtail bars have had strain gauges attached to measure the actual steel stress.

The fact that these tests were accepted in the past should not be taken as sufficient verification of acceptable performance in the future.

Among the critical shortcomings of at least some of the previous tests, have been the inconsistent support and loading conditions, where one or more of the following may have occurred:

- The supports at each end had been fixed, and could develop full friction. This means that compression stresses can develop in the base of the nib where it joins the web. A true roller support would make a considerable difference to the stress condition in the nib.

- The load application was too close to the support, either so close as to directly arch to the support, or close enough to provide confinement to the anchorage of the horizontal leg of the pigtail bar.

The first, in particular, makes a significant difference, and may go a long way to explaining the generally satisfactory performance of the pigtail in testing. Consider Figure 3 below. As the unit is loaded, the curvature will result in friction ($\mu N$) developing on the lower surface. Although NZS 3101 gives values of friction of 0.7 and 1 for concrete-steel and concrete-concrete respectively, it must be remembered that these are design figures, and therefore are lower-bound values. The in-service values of friction can be more than twice the design values.

![Figure 3: Nib reactions](image)

**FLEXURAL ANALYSIS OF THE NIB**

Using the loads described in the BRANZ report, we can calculate the stress at the bottom of the nib. Note that this form of elastic analysis is generally appropriate only for undisturbed (Bernoulli) zones. Clearly this is not really the case, but it is used here in a comparative sense to illustrate the influence of friction, and to demonstrate to an extent why testing may not have been successful in revealing the ultimate limit state performance of the units.

1. Untopped unit

The nib was analysed using the following assumptions:

- effective width of the flange, $b_{eff} = 0.2m$,
- coefficient of friction, $\mu = 1$
- eccentricity $e = 0.05m$
- Flange thickness $t = 0.05m$
- And the section modulus is: $z = \frac{b_{eff} t^2}{6}$

And therefore the stress on the lower surface of the nib,

$$fb = \frac{N.e}{z} + \frac{\mu N}{b_{eff} t} + \frac{\mu N \cdot t}{2z}$$

Or, $fb = N \left[ \frac{-e + \mu}{z} + \frac{\mu N \cdot t}{2z} \right]$
The BRANZ report lists load combination assumptions as follows:

\[ 1.2G + 1.5Q = 60\text{kN} \]
\[ 1.0G + 0.4Q = 28.1\text{kN} \]

This allows us to calculate the dead and imposed loads:

\[ G = 17.7\text{kN} \quad \text{and} \quad Q = 25.9\text{kN} \]

Substituting these loads and combinations, and using a concrete strength:

\[ f'_{c} = 77\text{MPa} \]

we can tabulate the bottom stress as follows (note that \(-ve\) stress = tension, and \(-ve\) coefficient of friction is acting towards the unit):

Table 1 : Untopped unit, friction \(\mu=-1\)

<table>
<thead>
<tr>
<th>Combination</th>
<th>N (kN)</th>
<th>(f_b) (MPa)</th>
<th>(f_b/f'_c)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(G + 0.4Q)</td>
<td>28.1</td>
<td>-5.62</td>
<td>0.07</td>
</tr>
<tr>
<td>(G + Q)</td>
<td>43.6</td>
<td>-8.73</td>
<td>0.11</td>
</tr>
<tr>
<td>(1.2G + 1.5Q)</td>
<td>60</td>
<td>-12</td>
<td>0.16</td>
</tr>
</tbody>
</table>

NB: NZS 3101 gives rupture stress:

\[ f_{r} = 0.6\sqrt{f'_{c}} \quad (= 5.3\text{MPa}) \]

However, it should be noted that the value in NZS 3101 is a lower bound value to be used for design. The tensile strength of concrete is highly variable, and values in excess of the required rupture stress determined above are understood to be common.

If, on the other hand, the coefficient of friction is first zeroed, and then reversed, we find that with all of the other assumed values remaining the same, the stress changes as in Tables 2 & 3 respectively:

Table 2 : Untopped unit, friction \(\mu=0\)

<table>
<thead>
<tr>
<th>Combination</th>
<th>N (kN)</th>
<th>(f_b) (MPa)</th>
<th>(f_b/f'_c)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(G + 0.4Q)</td>
<td>28.1</td>
<td>-16.9</td>
<td>0.22</td>
</tr>
<tr>
<td>(G + Q)</td>
<td>43.6</td>
<td>-26.2</td>
<td>0.34</td>
</tr>
<tr>
<td>(1.2G + 1.5Q)</td>
<td>60</td>
<td>-36</td>
<td>0.47</td>
</tr>
</tbody>
</table>

Table 3 : Untopped unit, friction \(\mu=-1\)

<table>
<thead>
<tr>
<th>Combination</th>
<th>N (kN)</th>
<th>(f_b) (MPa)</th>
<th>(f_b/f'_c)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(G + 0.4Q)</td>
<td>28.1</td>
<td>-28.1</td>
<td>0.36</td>
</tr>
<tr>
<td>(G + Q)</td>
<td>43.6</td>
<td>-43.6</td>
<td>0.57</td>
</tr>
<tr>
<td>(1.2G + 1.5Q)</td>
<td>60</td>
<td>-60</td>
<td>0.78</td>
</tr>
</tbody>
</table>

From this simplistic analysis, it can be seen that the friction is critical. With friction acting into the tee (as might be expected from testing with fixed restraints), the stress at the bottom of the flange is low enough that the concrete is unlikely to crack. But when the friction is reversed (as might be expected under seismic induced rotations or beam elongation), the unit is more likely to crack (Table 3).

Note that the assumption on the width of the effective section matching the web width only is quite conservative. A more reasonable width of say 400mm will obviously halve the stresses. However, use of the lower number allows for the possibility of a crack at the junction of the web and the rib, a common occurrence.

2. Topped unit

When the topping is added, the stresses change even more dramatically. If the thickness is amended to:

\[ \text{Flange thickness} \quad t = .115m \]

Then we can re-analyse the section with the following results:

Table 4 : Topped unit, friction \(\mu=0\), \(e=.05m\)

<table>
<thead>
<tr>
<th>Combination</th>
<th>N (kN)</th>
<th>(f_b) (MPa)</th>
<th>(f_b/f'_c)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(G + 0.4Q)</td>
<td>28.1</td>
<td>-3.19</td>
<td>0.04</td>
</tr>
<tr>
<td>(G + Q)</td>
<td>43.6</td>
<td>-4.95</td>
<td>0.06</td>
</tr>
<tr>
<td>(1.2G + 1.5Q)</td>
<td>60</td>
<td>-6.81</td>
<td>0.09</td>
</tr>
</tbody>
</table>
Table 5: Topped unit, friction \( \mu = -1, e = 0.09 \text{m} \)

<table>
<thead>
<tr>
<th>Combination</th>
<th>( N ) (kN)</th>
<th>( f_b ) (MPa)</th>
<th>( f_b/f'_c )</th>
</tr>
</thead>
<tbody>
<tr>
<td>G + 0.4Q</td>
<td>28.1</td>
<td>-10.62</td>
<td>0.14</td>
</tr>
<tr>
<td>G + Q</td>
<td>43.6</td>
<td>-16.5</td>
<td>0.21</td>
</tr>
<tr>
<td>1.2G + 1.5Q</td>
<td>60</td>
<td>-22.68</td>
<td>0.29</td>
</tr>
</tbody>
</table>

Note that in Table 4, the eccentricity was retained at 50mm, being the detailed eccentricity to the centre of the bearing from the root of the web. This reflects the service bearing condition, assuming a true roller support. Table 5 uses an eccentricity of 90mm, reflecting the case of the load applied at the end of the nib, as might be expected with support rotation and/or parallel beam elongation. Under the G+0.4Q load case, the tensile stress is still only 0.14\( f'_c \).

The significance of these results is that it is reasonably conceivable that test units might not crack under the test loads. Although all the tees that have been previously tested may not have reached the same concrete strength as the unit that BRANZ tested, it is likely that they will have been of a reasonably high strength.

It follows that the pigtail itself is not being tested under these circumstances, where the tee either does not crack, or is not pre-cracked to ensure that the steel of the pigtail is stressed. Hence, the SESOC recommendation that the units are tested under the third regime described above, ie research testing in support of the development of a rational design method. Although a shear-friction model may have given similar results to the test loads, it is not appropriate, as shown above.

**DESIGN REQUIREMENTS**

It is not sufficient to simply consider the gravity load in the design of flooring systems. Although a floor's principal function may be to support the dead and imposed vertical loads, it must be able to maintain that support through the range of other events that it may be subjected to over its life.

The main load condition that designers in New Zealand face, other than gravity, is earthquake. For many years we have been familiar with capacity design principles and able to design structures that could deform well beyond their initial yield strength. But the wider implications of that have not always been so well understood. More recent research and code developments have added to our understanding of deformation compatibility.

In designing a ductile structure, a Design Basis Earthquake (DBE) load is established, depending on the ground conditions, structure type, location, and building importance level. For an average building, of importance level 2, we use a 500 year return period earthquake as our DBE, and it is this that we use as our ultimate limit state (ULS) design level. There is also an explicit serviceability limit state (SLS) condition that must be checked.

But beyond those explicit verifications, there is an implicit expectation that the building will not collapse in the Maximum Credible Earthquake (MCE), nominally a 2500 year return period earthquake. While this is not checked specifically, the material standards such as NZS 3101 act as ‘deemed to comply’ methods of achieving this performance. So although the drift limit for example is 2.5%, there is an expectation that the building will have the ability to deform to about 4.5%, if the detailing provisions are correctly followed.

In elements such as the precast flooring systems, the MCE expectation will not necessarily be met unless specifically checked as there are not specific provisions that cover that in the standard. Therefore a designer must at least consider the implications of the increased drifts in determining whether a system is appropriate for use in a given situation. Often this may be a trivial check, but in the case of details that may have a finite deformation limit, it may be critical. Failure of floor systems to sustain load to the MCE drift limit could initiate gross collapse of significant areas of buildings, with consequent loss of life.

In addition, floors generally have an additional function during an earthquake, of distributing seismic loads to the primary load resisting elements, by diaphragm action. The forces generated in a diaphragm can be significant, and add additional stress at locations such as supports. While the design of the diaphragm must be a primary responsibility of the building designer, the precast floor designer should be aware of the implications of this.

The SESOC article covers the range of load and deformation considerations, so the full list will not be repeated here. But in summary, designers need to consider the full range of actions, both loads and deformations, which a floor system may be subjected to, including the MCE event.

**DESIGNERS’ RESPONSIBILITIES**

The SESOC paper presented a section detailing the responsibilities of both the designers and the precasters. Since the publication of the first draft of the SESOC paper, DBH has published its draft guidelines for the design, assessment and retrofit
of hollowcore flooring systems\textsuperscript{10}. Although there are some variations in detail, the two documents are generally in agreement.

The designer has the primary responsibility for the performance of the flooring system. Although the floor may be sourced as a design and supply item under a performance specification, the designer still has an obligation to determine that any such elements meet the specification that is set.

It is worth repeating what has already been published in the SESOC paper, as a reasonable guide as to who should do what:

The structural engineer should be responsible for:

- Specifying a flooring system that can meet the functional brief with regard to live loads, thermal and shrinkage movements, seismic performance (with respect to both diaphragm requirements and elongation and rotation at the supports), durability and fire
- Specifying a support method for the flooring that can satisfy both long-term and short-term loading provisions as well as secondary effects. This must include consideration of the DBE and MCE requirements as noted above.
- Particular attention is required to provide adequate shear strength of units near supports under seismic actions in negative moment regions (see NZS 3101:2006, including Amendment 2 cl. 19.3.11.2.4)
- Providing (in the specification and/or on the drawings) sufficient information on loadings and potential movement, that the precaster can complete the detailed design of the units, including the end seating condition (e.g. flange hung nib). This should include any secondary actions that the precaster (or designer of the units) needs for the verification of the design, such as beam elongations (in the case of seismic frames), support rotations, and thermal actions.
- Reviewing the precaster’s design for response to the brief (note that the review is generally NOT expected to alleviate the precaster of any responsibility in respect of the design of the units).
- Construction monitoring of the structure, verifying that the general intent of the structural drawings is satisfied, i.e. that all insitu concrete, reinforcement and other embedded items are constructed on site generally as detailed.

The precaster should be responsible for:

- Verifying that the nominated floor system is appropriate for the specified loads and actions.
- Design of the floor units including the supports (e.g. nibs, daps – only the part that is integral with the precast flooring unit, not the supporting structure). The design of the units may be by conventional engineering design to the prevailing standards (typically NZS 3101, or alternative solution where it has been verified that this will be acceptable to both the structural engineer and the relevant Territorial Authority (TA)), or by test (where the testing regime is in compliance with Appendix B of AS/NZS 1170 as described here). In either case, the design offered must address the movements and rotations at the support as well as the basic loading information provided by the Structural Engineer.
- Provision of sufficient information for the structural engineer to review the design of the precast floor prior to final submission to the TA for a full Building Consent (note that the requirement for further review of the flooring systems may vary between TA’s but in principle all proprietary items require final review before the Building Consent can be fully signed off), This should include the provision of a Producer Statement PS-1 by the precaster’s design engineer.
- Quality Assurance testing and review of the manufacture of the units such that the precaster can support the contractor’s provision of a PS-3 for final sign-off of the completed structure.

THE WAY FORWARD

As a proprietary item, it is not the responsibility of SESOC to complete the re-design of the pigtail. In a commercial environment, manufacturers will determine which systems best suit their production facilities or offer them a competitive advantage. However, SESOC can advise on its opinion of the compliance or otherwise of details, and inform its members and the wider design fraternity of the implications of use of particular details.

In respect of the pigtail, SESOC’s advice has been unequivocal. Either the detail must be modified to comply with NZS 3101, or the testing programme that is followed must develop a rational design methodology that can be verified independently to be a suitable alternative design method. That has not happened to date.
At a recent meeting, SESOC representatives discussed some of the areas of non-compliance and made suggestions as to possible ways to deal with these, but it is now with the manufacturers to determine what modifications they wish to make. Some of the non-compliance issues are undoubtedly the result of manufacturing issues that SESOC is not party to, so whether modification is practical or not is something that we cannot advise on.

However, with the safety of the public potentially at stake, the situation is clear – knowing what we now know, we cannot use the detail until these matters are resolved.

There remains the question over the safety of existing installations. This is now a matter for consideration of the Department of Building and Housing as well as designers and manufacturers. At the least, a building must comply with the Building Act, Section 122, covering earthquake prone buildings. Broadly, a building must be able to resist 33% of current code actions to comply. While this intuitively seems likely, further research is required to verify this. This will take some time, and could be an outcome of any testing done by Precast New Zealand and the manufacturers, but would probably need to be done in a more ‘investigative’ fashion than most of the testing to date.

ACKNOWLEDGEMENTS

The author would like to thank the other members of the study group, in particular Richard Fenwick for his patient re-education of the author on the fundamentals of concrete shear mechanisms and strut and tie models. Also to Ashley Smith, who has contributed significantly to the process.

REFERENCES

1. SESOC News No 4, September 2008, SESOC Website.


4. Chartered Professional Engineers of New Zealand Act 2002

5. NZ Building Code Clause B1 Structure, DBH, July 1992


9. BRANZ Test ST0752, March 20, 2009, Beattie Cyclic Displacement of a 300 Deep Double Tee floor unit to simulate frame extension under earthquake loading

10. Precast Floor Overview Group (PCFOG), comprising representatives of SESOC, NZSEE, NZCS, Preliminary Draft – April 2009, Seismic Performance of Hollow Core Floor Systems, DBH

11. Building Act 2004