An Independent Analysis Of NISTs Scientific Methods And Assumptions

Scott Jones BSc (scottjones05@aol.com)

Summary of Key points

NISTs official report says global collapse is inevitable following the establishment of the conditions for the initiation of collapse. This is dependent on a 2002 paper proposing a mechanism for progressive collapse of a steel structured building. However the 2002 paper also clearly states that it is based on a hypothesis that the upper section is rigid at the instant of impact with the lower section and makes assumptions of the mass of the upper section, the design load capacity of the lower section and the stiffness of the structure to calculate the overload ratio. Using data in NISTs report and information available online it can be shown that there were major errors in these assumptions and in fact the overload ratio was less than 1 for both the north and south towers. As every explanation of how the supporting structure collapsed relies on this overload ratio then collapse was not inevitable, as NIST state, rather it was unlikely without some extra action or system to remove the supporting structure, or extra mass to overload the supporting structure.

Supporting Argument

NIST's "Final Report of the National Construction Safety Team on the Collapses of the World Trade Center Tower" (hereafter referred to as "The Final Report") states its first objective was to "Determine why and how WTC 1 and WTC 2 collapsed following the initial impacts of the aircraft"[1] (emphasis mine). To determine this The Final Report needs to explain the events following the initial impact until the collapse of the buildings is complete. NIST state that their approach was to simulate the behaviour of the tower using "four steps:

1. The aircraft impact into the tower, the resulting distribution of aviation fuel, and the damage to the structure, partitions, thermal insulation materials, and building contents.
2. The evolution of multi-floor fires.
3. The heating and consequent weakening of the structural elements by the fires.
4. The response of the damaged and heated building structure, and the progression of the structural component failures leading to the ***initiation*** of the collapse of the towers."[2] (emphasis mine)

Their approach appears to fall short of that which would be required to determine why and how the towers collapsed as it stops at the initiation of collapse, not at the completion of it. There is no explicit explanation why they took this approach in The Final Report or in any of the supporting documents, however their footnotes state that they conducted "little analysis of the structural-behaviour of the tower after the conditions for collapse were reached and collapse became inevitable."[3] (emphasis mine – NOTE: in another footnote (footnote 13, page 82), "conditions for collapse" is replaced with "conditions for collapse initiation", these are one and the same). Although I can find no analysis in the Final Report or supporting documents of the structural behaviour of the tower after collapse initiation, this sentence implies they have made an assumption, or have unstated proof, that once the collapse begins global collapse cannot be avoided. There is no justification offered for why this assumption was made and no evidence or sources given to support its validity.

However, elsewhere in The Final Report they state that their recommendation for further research into the "Prevention of progressive collapse" is "related" to 9/11[4] (emphasis mine). The Final Report also states that "Progressive collapse (or disproportional collapse) occurs when an initial local failure spreads from structural element to structural element resulting in the collapse of an entire structure or a disproportionately large part of it."[5] From this statement it can be inferred that "the conditions for collapse" were defined as the initial conditions for progressive collapse, which The Final Report states to be "an initial local failure [that] spreads from structural element to structural element". To check the validity of this definition it is necessary to examine the peer reviewed scientific papers modelling progressive collapse to see whether this is true.

Reading the peer reviewed work on the subject of progressive collapse reveals that there had, at that time, only ever been one paper modelling the mechanics of the collapse of the twin towers after its initiation, a paper called "Why Did the World Trade Center Collapse? – Simple Analysis" by Zdenek P. Bazant, Fellow ASCE, and Yong Zhou (hereafter referred to as "The Paper"). Almost all other papers on the subject reference this paper directly (including Bazant authored papers on the subject) and use the overload ratio within as a basis for further analysis without evidence of recalculation or questioning of the assumptions. Or they state "it has already been well established that at the moment of impact the supporting structure in the lower section is overcome by an order of magnitude" (paraphrased) i.e. the overload ratio in The Paper is correct in all circumstances. Therefore if The Paper supports NISTs definition then it can be assumed to be valid and hence NISTs conclusions are valid. [NOTE: the 2007 paper authored by Seffen does propose an alternative model for the collapse after it has begun, but still states the overload ratio is correct without recalculation – see 'Selected Supporting Evidence' for details]

Using a simplified model the paper presents an analysis of the overall collapse of the twin towers and claims that the towers were doomed "if prolonged heating caused the majority of columns of a single floor to lose their load carrying capacity"[6]. However, as stated in The Paper "an important hypothesis implied in this analysis is that the impacting upper part, many floors in height, is so stiff that it does not bend nor shear on vertical planes, and that the distribution of column displacements across the tower is almost linear, like for a rigid body. If, however, the upper part spanned only a few floors (say, 3 to 6), then it could be so flexible that different column groups of the upper part could move down separately at different times, producing a series of small impacts that would not be fatal (in theory, if people could have escaped from the upper part of the tower, the bottom part of the tower could have been saved if the upper part were bombed, exploded or weakened by some 'smart' structure system to collapse onto the lower part gradually as a mass of rubble, instead of impacting it instantly as an almost rigid body)."[7] The paragraph is quoted in full as it has two implications directly linked to NISTs definition of "the conditions of collapse".

The first implication is that, despite admitting that the flexibility of the upper section is dependent on the number of floors, and that it admits the upper section "could be so flexible that different column groups of the upper part could move down separately at different times, producing a series of small impacts that would not be fatal" no justification is offered as to where the threshold of flexibility is. Hence the assumption that 15 floors "is so stiff that it does not bend or shear on vertical planes, and that the distribution of column displacements across the tower is almost linear, like for a rigid body" must be proven to be true for the world trade centre towers before this paper can be proved to explain the towers complete collapse after initiation of collapse. [NOTE: In their addendum to The Paper Bazant and Zhou state that the reason the upper section was rigid in the case of the north tower was that the ratio between width and height was 0.7, however this neglects the potential effect of the fires]

The second implication is that, as "<u>the bottom part of the tower could have been saved if the upper part were</u> bombed, exploded or <u>weakened by some 'smart' structure system to collapse onto to the lower part gradually as a mass of rubble, instead of impacting it instantly as an almost rigid body</u>", then it must be shown conclusively that the 'system' visible to the naked eye, namely rising heat from the fires started by the jet fuel, did not weaken the upper section so it could break it into smaller sections before it impacts.

<u>Rigidity</u>

The importance of whether the upper section is rigid or not at the point of impact can be easily demonstrated by visualising a simple model. Consider a solid tower with a density equal to that of sand. If we drop a solid object with the same density and a mass of say, 1 Kg, then it is easy to see how the mechanism proposed in The Paper applies (shown in Fig. 1 in The Paper[10]), however if we allow 1 Kg of sand to fall on top of the same tower then it is clear that mechanism proposed in the paper does not apply.

The question of whether the upper section of the tower was rigid at the instant of impact can also be seen when comparing the diagrams in Fig. 1.3, 1.4 and 1.5 from The Paper[9] showing the stages of 'progressive collapse' with the visual record:

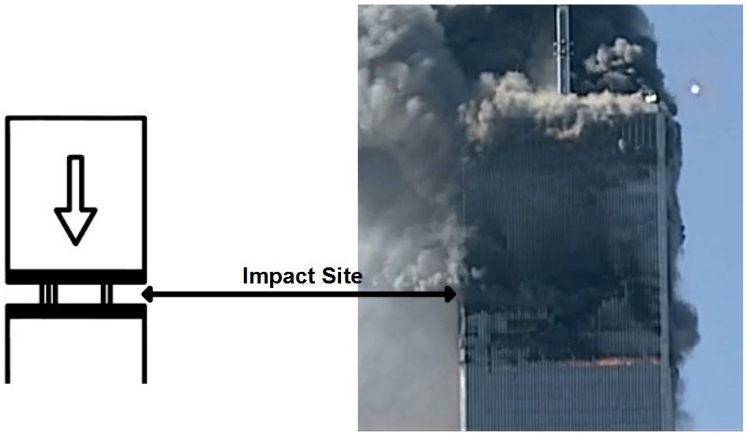

Fig. 1.3

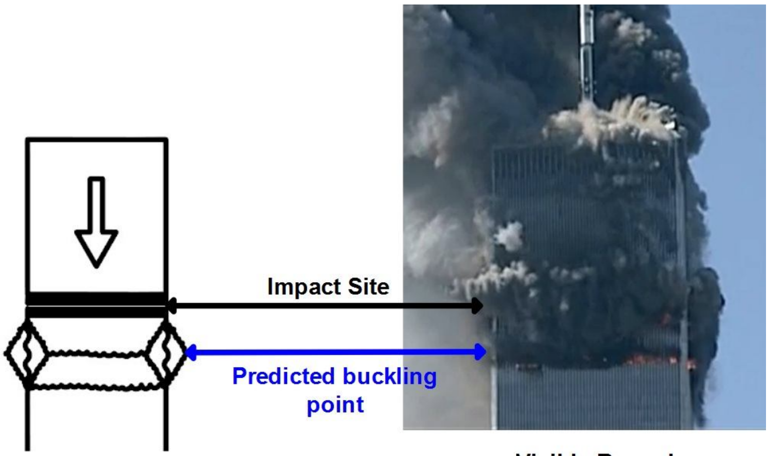

Fig. 1.4

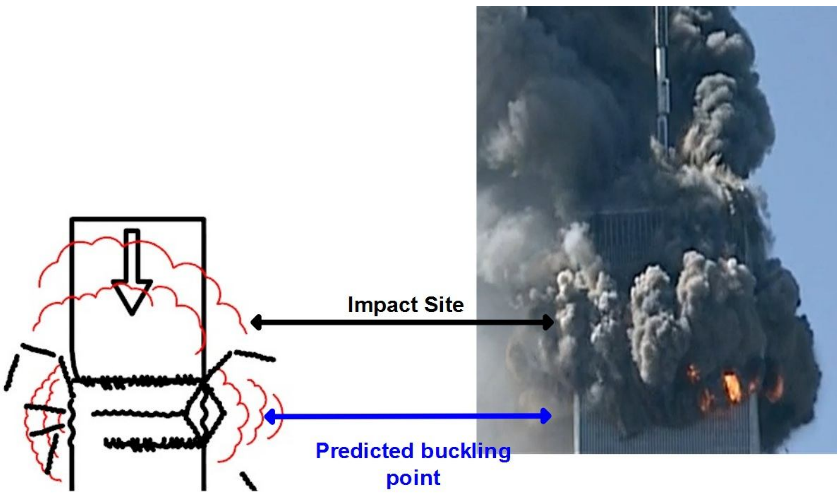

Fig. 1.5

Overload Ratio

The reason the rigidity of the upper section is important is the calculation of the overload ratio in the The Paper, calculated on page 2. A glance at the variables shows that it is dependent on the mass at the instant of impact (assumed to be approximately 58000 tonnes), if the upper section was broken into, say, 3 identical pieces then the overload ratio would have to be calculated for an impact of 19333 tonnes, then another, separate 19333 tonnes, and then again.

More specifically The Paper states that the maximum force applied by the upper part, P, can be calculated using the equation:

$$mg\left(h + \frac{P}{C}\right) = \frac{P^2}{2C}$$

Where $h = 3.7m$ is the height of the critical floor columns, $C \approx 71$ GN/m is the stiffness of the structure beneath, $g = 9.8m/s^2$ is acceleration due to gravity, and $m \approx 58 \times 10^6$ kg is the mass of the falling upper section, if m was lower then all further conclusions drawn are merely one of a range of possibilities, until the exact value(s) of m can be determined.

From this quadratic the paper states that the overload ratio can be calculated with the following

$$\frac{P_{dyn}}{P_0} = 1 + \sqrt{1 + \frac{2Ch}{mg}}$$

This arises as the values of m in P_{dyn} ($= P_{max}$) and P_0 are assumed to be the same, however this is contradicted by the construction details NIST provide and to calculate the overload ratio it is necessary to calculate P_{dyn} and P_0 separately. In addition, although g is a constant in freefall, the stiffness has no stated error margin and no supporting evidence given for the accuracy of the value. To determine the actual overload ratio it is necessary to calculate these values as accurately as possible.

Firstly, the value of P_0, the design load capacity (which was approximated as the minimum required to support the mass of the floors above, mg) is far below the estimates that can be calculated from NISTs own research. Research conducted by others states that:

"One aspect of engineering that is not widely understood is that structures are over-engineered as a matter of standard practice. [7] Steel structures like bridges and buildings are typically designed to withstand five times anticipated static loads and 3 times anticipated dynamic loads. The anticipated loads are the largest ones expected during the life of the structure, like the worst hurricane or earthquake occurring while the floors are packed with standing-room-only crowds. Given that September 11th was not a windy day, and that there were not throngs of people in the upper floors, the critical load ratio was probably well over 10, meaning that more than nine-tenths of the columns at the same level would have to fail before the weight of the top could have overcome the support capacity of the remaining columns.

There is evidence that the Twin Towers were designed with an even greater measure of reserve strength than typical large buildings. According to the 1964 white paper cited above, a Tower would still be able to withstand a 100-mile-per-hour wind after all the perimeter columns on one face and some of the columns on each adjacent face had been cut. [8] Also, John Skilling is cited by the *Engineering News Record* for the claim that "live loads on these [perimeter] columns can be increased more than 2000% before failure occurs." [9]

7. Factor of safety, *StateMaster.com*, [cached]
8. City in the Sky, *Times Books ...*, , page 133
9. How Columns Will Be Designed for 110-Story Buildings, *ENR*, 4/2/1964". Quoted from http://911research.wtc7.net/wtc/analysis/design.html .

Reviewing The Report and supporting documents we can obtain a more reasonable estimate for the overload ratio.

Factor of Safety (FoS) = 2 (approximate FoS for steel structures)

Highest estimate of tower mass = 500,000,000kg (Ashley, Eagar and Musso, Tyson, actual figure probably below 300,000,000 kg according to G. H. Ulrich)

Reduced design live load capacity = 50 pounds per square foot (NIST-NCSTAR 1-2a, page)

Estimated live load on 9/11 = 12.5 psf (NIST-NCSTAR 1-6 page 73)

Floor area = 40000 sq. Feet (NIST-NCSTAR 1-2B page 53 Table 3-1)

Mass of upper section (assuming it was rigid):

Mass of 12 floors = 54,500,000kg ($\frac{12}{110}$ of total mass)

1 floor load = 500,000 lbs (estimated load capacity per square foot x floor area)

12 floors load = 6,000,000 lbs = 2,700,000 kgs

Total mass of upper section = 57,200,000 kg = 0.0572 Tg (conversion to Teragrams due to stiffness being given in GN/m)

Design load capacity:

13 floors dead load = 59,100,000 kg (13 floors to account for missing floor)

1 floor reduced live load capacity = 2,000,000 lbs (reduced design live load capacity x floor area)

13 floors reduced design live load capacity = 26,000,000 lbs = 11,900,000kg

Total design load (incl. FoS) = 142,000,000kg = 0.142 Tg ($m \times FoS$)

Using these estimates (and the quadratic formula) to calculate the overload ratio we get:

$$P_{dyn} = mg + \sqrt{m^2g^2 + 2Cmgh} \approx 17.7$$

$$P_0 = 0.142 * 9.8 \approx 1.4$$

$$\frac{P_{dyn}}{P_0} \approx 12.7$$

Searching NISTs supporting documents it is possible to gain a more accurate measurement of the mass involved in the collapse and refine the overload ratio further (see Appendix A).

From this data floor 103 has mass of approximately 2,240,000 kg, which agrees reasonably well with NISTs own figure of 2,000,342 kg (NIST-NCSTAR 1-2B page 53). As tenant floors 103 and down are the same we can now use these estimates of the mass of the upper section and design load.

Upper 12 floors ≈ 31,990,000 kg = 0.03199 Tg

Upper 13 floors design load (incl. FoS) ≈ 94,470,000 kg = 0.09447 Tg

$$P_{dyn} \approx 13.2$$

$$P_0 \approx 0.926$$

$$\frac{P_{dyn}}{P_0} \approx 14.2$$

Which is slightly higher than the estimated value previously, but if we take these new figures and apply them to south tower we get a surprising result.

Upper 29 floors ≈ 70,060,000 kg = 0.07006 Tg

Upper 30 floors design load (incl. FoS) ≈ 199,160,000 kg = 0.19916 Tg

$$P_{dyn} \approx 19.7$$

$$P_0 \approx 1.95$$

$$\frac{P_{dyn}}{P_0} \approx 10.1$$

This counter intuitive result for the south tower implies that there was considerably more structural strength in the north tower than its overload ratio shows, possibly due to the unknown axial design load for wind resistance. However the stiffness should also vary with height and, after a thorough search we find NISTs elastic modulus of 206 GPa (Figure 7-1 NIST-NCSTAR 1-3 page 104) and cross section diagrams of almost all core columns, which were extracted from data released under a freedom of information act request. Using this data, the formula for axial stiffness and Hooke's Laws it is possible to model each section of each column of the core and perimeter as springs calculate the stiffness accurately (see Appendix B for details).

These more accurate calculations show the stiffness was massively overestimated in The Paper, the actual stiffness at floor 97 (95 for simplicity) was approximately 4.4 GN/m, in the south tower at floor 80 the stiffness was approximately 6.2 GN/m. Reading The Paper it seems the error was in assuming that the columns didn't reduce in mass with height.

We can also use a better estimate of the factor of safety, the core and perimeter columns shared the gravity load unevenly, 53% and 47% respectively, however the perimeter wall was also designed to resist wind loads and it's factor of safety was much higher, 5.7 according to NIST[15]. If we share the design loads between the perimeter and core columns in the correct proportion, apply the factor of safety separately and then recalculate the total design loads we arrive at the figures of approximately 0.35823 Tg for the north tower and 0.75521 Tg for the south tower.

Using these new figures we can calculate the overload ratios again to see if global collapse was inevitable (in these calculations the height of one floor is taken as the accurate measure of 3.6575m).

North Tower	South Tower
1 floor drop, Core FoS = 2.1, Perimeter FoS = 5.7 C ≈ 4.4 GN/m	1 floor drop, Core FoS = 2.1, Perimeter Fos = 5.7 C ≈ 6.2 GN/m
Upper 12 floors ≈ 31,990,000 kg = 0.03199 Tg Upper 13 floors design load (incl. FoS) ≈ 0.35823 Tg $(0.53 \times m \times 2.1 + 0.47 \times m \times 5.7)$ $P_{dyn} \approx 3.495$ $P_0 \approx 3.511$ $\frac{P_{dyn}}{P_0} \approx 0.996$	Upper 29 floors ≈ 70,060,000 kg = 0.07006 Tg Upper 30 floors design load (incl. FoS) ≈ 0.75521 Tg $(0.53 \times m \times 2.1 + 0.47 \times m \times 5.7)$ $P_{dyn} \approx 6.291$ $P_0 \approx 7.401$ $\frac{P_{dyn}}{P_0} \approx 0.849$

If also we take into account the following factors:

1. The visual record doesn't support the assumption of free fall for one full floor.
2. The visual record shows a considerable amount of material in between the upper section and the uppermost intact floor of the lower section, which would have provided resistance and reduced the acceleration of the upper section.
3. John Skilling is cited by the *Engineering News Record* for the claim that "live loads on these [perimeter] columns can be increased more than 2000% before failure occurs." (How Columns Will Be Designed for 110-Story Buildings, *ENR*, 4/2/1964), meaning the actual factor of safety for the perimeter columns, and by extension the core columns, were almost certainly higher than 5.7 and 2.1 respectively.
4. That design load is the ***minimum point*** at which failure *could* occur.

It can clearly be seen that not only was collapse not inevitable as NIST state, it was improbable in the case of both towers. It is worth noting that heat in the core and perimeter columns is irrelevant as it would decrease stiffness as well as reducing load capacity, and all the impact and fire damage was on the impact site and above.

At this point it is also worth considering the second overload ratio given the paper, derived from the elastic wave equation. This overload ratio is valid for a maximum of $\frac{2}{1000}$ths of a second after the impact of the upper section, while the elastic wave generated is propagating to the ground. During this time the upper section moves a maximum of 2cm downwards and so its effects can be ignored for the purpose of determining whether the structure below could have collapsed. Analysis of the overload ratio of 64.5 in The Paper reveals that the cross sectional stiffness – that being the stiffness before division by length – used by the paper was ten times more than the average cross sectional stiffness calculated from actual cross section diagrams (approximately 16000 GN compared to approximately 1600 GN).

Selected Supporting Evidence

In their supporting documents[11] NIST do discuss other collapse hypotheses, The Paper already mentioned, Weidlinger Associates 2002 study which was to show that the two collapses were independent of each other and deals only with the collapse initiation, Maryland which they state they disagree with and two studies from Edinburgh and Arup, both of which deal with a cross section of floors and so don't impact the overload ratio calculation in The Paper. Hence my assertion that the only justification for their statement that collapse was inevitable is The Paper.

Quotes from other papers on the subject include:

> "To explain the collapse, it was proposed (on September 13, 2001; Bazant 2001; Bazant and Zhou 2002) that viscoplastic buckling of heated and overloaded columns caused the top part of tower to fall through the height of at least one story, and then shown that the kinetic energy of the impact on the lower part must have exceeded the energy absorption capacity of the lower part by an order of magnitude." - What Did and Did not Cause Collapse of WTC Twin Towers in New York (Bazant, LE, Greening and Benson, Journal of Engineering Mechanics ASCE, Vol 134 2008)
>
> "The subsequent near free-falling of these upper parts over the height of just one storey resulted in dynamical "over-loading" of the relatively undamaged lower columns by a factor of 30 compared to their static load capacity, according to Bazant and Zhou (2002)" and
>
> "The resulting impingement produced peak forces correctly identified by Bazant and Zhou (2002) to be far in excess of the design capacity of these columns and hence, above the expected value of P(max) (Fig. 1) that could be reasonably carried by them, even if perfect and undamaged. These columns began to deform plastically, thereby seeding failure of this, next part of the structure."– Progressive Collapse of the World Trade Center: a Simple Analysis (Seffen, 2007)
>
> "Regardless of the load capacity of the columns, there is no way to deny the inevitability of progressive collapse driven by gravity *alone* if this criterion is satisfied (for the World Trade Center it is satisfied with an order-of-magnitude margin)" Mechanics of Progressive Collapse: Learning from World Trade Center and Building Demolitions (Bazant and Verdure, 2007)

The Final Reports flow chart[12] also shows that only three of their studies fed into the global collapse scenario, NCSTAR 1-2, NCSTAR 1-3, and NCSTAR 1-6, none of which recalculate the overload ratio or discuss any events after collapse had initiated.

> "**1. Was there enough gravitational energy present in the World Trade Center Towers to cause the collapse of the intact floors below the impact floors? Why was the collapse of WTC 1 and 2 not arrested by the intact structure below the floors where columns first began to buckle?**
>
> Yes, there was more than enough gravitational load to cause the collapse of the floors below the level of collapse initiation in both WTC Towers. The vertical capacity of the connections supporting an intact floor below the level of collapse was adequate to carry the load of 11 additional floors if the load was applied gradually and 6 additional floors if the load was applied suddenly (as was the case). Since the number of floors above the approximate floor of collapse initiation exceeded six in each WTC Tower (12 and 29 floors, respectively), the floors below the level of collapse initiation were unable to resist the suddenly applied gravitational load from the upper floors of the buildings. Details of this finding are provided below:
>
> Consider a typical floor immediately below the level of collapse initiation and conservatively assume that the floor is still supported on all columns (i.e., the columns below the intact floor did not buckle or peel-off due to the failure of the columns above). Consider further the truss seat connections between the primary floor trusses and the exterior wall columns or core columns. The individual connection capacities ranged from 94,000 lb to 395,000 lb, with a total vertical load capacity for the connections on a typical floor of 29,000,000 lb (See Section 5.2.4 of NIST NCSTAR 1-6C). The total floor area outside the core was approximately 31,000 ft^2, and the average load on a floor under service conditions on September 11, 2001 was 80 lb/ft^2. Thus, the total vertical load on a floor outside the core can be estimated by multiplying the floor area (31,000 ft^2) by the gravitational load (80 lb/ft^2), which yields 2,500,000 lb (this is a conservative load estimate since it ignores the weight contribution

of the heavier mechanical floors at the top of each WTC Tower). By dividing the total vertical connection capacity (29,000,000 lb) of a floor by the total vertical load applied to the connections (2,500,000 lb), the number of floors that can be supported by an intact floor is calculated to be a total of 12 floors or 11 additional floors.

This simplified and conservative analysis indicates that the floor connections could have carried only a maximum of about 11 additional floors if the load from these floors were applied statically. Even this number is (conservatively) high, since the load from above the collapsing floor is being applied suddenly. Since the dynamic amplification factor for a suddenly applied load is 2, an intact floor below the level of collapse initiation could not have supported more than six floors. Since the number of floors above the level where the collapse initiated, exceeded 6 for both towers (12 for WTC 1 and 29 for WTC 2), neither tower could have arrested the progression of collapse once collapse initiated. In reality, the highest intact floor was about three (WTC 2) to six (WTC 1) floors below the level of collapse initiation. Thus, more than the 12 to 29 floors reported above actually loaded the intact floor suddenly." [NIST FAQ Supplement Dec 14 2007]

- This answer deals only with the floors, a version of the 'pancake' theory, it explains how it was possible for the floors to separate from the supporting structure but it doesn't explain how the supporting structure itself collapsed from just the falling mass of the upper section. It also doesn't discuss the beam supported mechanical floors and whether these were strong enough to arrest the 'pancake' collapse of the floors.

Conclusion

The assumptions in the calculation of the overload ratio in The Paper mean the definition of "the conditions for collapse" in The Final Report is incomplete and should have been "an initial local failure [that] spreads from structural element to structural element, and proof that the impact of the upper section of the tower on the lower section of the tower was sufficient to overcome the load capacity of the lower section".

Furthermore, if indeed there are no other demonstrable mechanisms for explaining the total collapse of the lower section of the towers then the inferred definition of is irrelevant. NIST is in error when it states collapse is inevitable from impact damage and fire alone as it is unlikely the falling mass of the upper section could overload the supporting structure beneath.

Footnotes

[1] Section E.1, page xxxv of The Final Report
[2] Section E.2, pages xxxvi and xxxvii of The Final Report
[3] Footnote 2, page xxxvii of The Final Report
[4] Table E-1, page xliv of The Final Report
[5] Footnote 19, page 206 of The Final Report
[6] page1 of The Paper (page is labelled XVIII in linked document)
[7] pages 2 and 3 of The Paper (pages are labelled XIX and XX in linked document)
[8] page 1 of "Addendum to "Why Did The World Trade Center Collapse? – A Simple Analysis"
[9] page 8 of The Paper (labelled XXV in linked document)
[10] page 8 of The Paper (page is labelled XXV in linked document)
[11] Section 9.4.4 beginning on page 322 of NIST NCSTAR 1-6D
[12] page xxxi of the Final Report
[13] Section E.3.1 beginning on page lii of NIST NCSTAR 1-6D
[14] section 9.4.5 beginning on page 326 of NIST NCSTAR 1-6D
[15] derived from values in NCSTAR 1-6 Figure 4-35 page 101, clarification provided to Mark H. Gaffney Dec 14 2006 see footnote 70 http://www.informationclearinghouse.info/article18999.htm for details

References

The Final Report: http://www.nist.gov/customcf/get_pdf.cfm?pub_id=909017

Supporting documents to The Final Report: http://www.nist.gov/el/disasterstudies/wtc/wtc_finalreports.cfm

NIST information on progressive structural collapse: http://www.nist.gov/el/topic_collapse.cfm

The Paper (Why Did The World Trade Center Collapse? – A Simple Analysis, Bazant And Zhou, 2001): http://www.mae.ncsu.edu/eischen/courses/mae543/docs/BazantWTC.pdf

Addendum to "Why Did the World Trade Center Collapse? – A Simple Analysis" (Bazant and Zhou, 2002): http://www.civil.northwestern.edu/people/bazant/PDFs/Papers/405.pdf

Selected list of Scientific papers on the progressive collapse of the twin towers: http://sites.google.com/site/wtc7lies/peer-reviewedpapersaboutthewtcimpacts,fi

FAQ: http://www.nist.gov/public_affairs/factsheet/faqs12007.cfm

Weidlinger Assocites Study: http://www.wai.com/articles_pdf/webAS_abboudlevy_wtc_asceforensic_2003.pdf

Mechanics of Progressive Collapse: Learning from World Trade Center and Building Demolitions (Bazant and Verdure, 2007): http://www.civil.northwestern.edu/people/bazant/PDFs/Papers/466.pdf

Progressive Collapse of the World Trade Center: a Simple Analysis (Seffen, 2007): http://winterpatriot.pbworks.com/f/seffen_simple_analysis.pdf

What Did and Did not Cause Collapse of WTC Twin Towers in New York (Bazant, LE, Greening and Benson, Journal of Engineering Mechanics ASCE, Vol 134 2008): http://www.civil.northwestern.edu/people/bazant/PDFs/Papers/00%20WTC%20Collapse%20-%20What%20Did%20&%20Did%20Not%20Cause%20It.pdf

Why Did the World Trade Center Collapse? Science, Engineering, and Speculation, *JOM (The Journal of the Minerals, Metals and Materials Society)*. 53(12), pp.8-11 Eagar, T.W. and Musso, C., (2001): http://www.tms.org/pubs/journals/JOM/0112/Eagar/Eagar-0112.html

When the Twin Towers Fell, Ashley, S., (October 09, 2001), *Scientific American*

Towers of Innovation, Tyson, P., *PBS/NOVA:* http://www.pbs.org/wgbh/nova/wtc/innovation.html

Analysis of the Mass and Potential Energy of the World Trade Centre Tower (Gregory H. Ulrich): http://www.journalof911studies.com/volume/200703/GUrich/MassAndPeWtc.pdf

WTC Modelling and Simulation: http://wtcmodel.wikidot.com/

9-11 Research: http://911research.wtc7.net/index.html

Still Dead On Arrival (Mark H. Gaffney): http://www.informationclearinghouse.info/article18999.htm

Appendix A – Mass of floors

Floor	Section	Section Area (sq. ft)	CDL Design (psf)	SDL NIST (psf)	LL Design (psf)	LL Red (%)	Red. LL (psf)	9/11 Load (psf)	Design Load (psf)	9/11 Load (kg)	Design Load (kg)
103	Long Span	9329[b]	50[d]	14[g]	100[i]	50[m]	70[n]	81.5[r]	134[s]	344872	567029
Tenant[a]	Short Span	4431[b]	42[e]	14[g]	100[i]	50[m]	82.5[n]	76.625[r]	138.5[s]	154006	278367
	Two Way	17497[b]	42[e]	16[g]	100[i]	50[m]	55[n]	71.75[r]	113[s]	569444	896825
	Core	8694[c]	250[f]	33[h]	100[i]	50[m]	56[o]	297[r]	339[s]	1171229	1336857
104	Long Span	9329[b]	50[d]	14[g]	100[i]	55[m]	70[n]	81.5[r]	134[s]	344872	567029
Tenant[a]	Short Span	4431[b]	42[e]	14[g]	100[i]	55[m]	82.5[n]	76.625[r]	138.5[s]	154006	278367
	Two Way	17497[b]	42[e]	16[g]	100[i]	55[m]	55[n]	71.75[r]	113[s]	569444	896825
	Core	8694[c]	250[f]	33[h]	100[i]	55[m]	56[o]	297[r]	339[s]	1171229	1336857
105	Long Span	9329[b]	50[d]	14[g]	100[i]	60[m]	70[n]	81.5[r]	134[s]	344872	567029
Tenant[a]	Short Span	4431[b]	42[e]	14[g]	100[i]	60[m]	82.5[n]	76.625[r]	138.5[s]	154006	278367
	Two Way	17497[b]	42[e]	16[g]	100[i]	60[m]	60[q]	73[r]	118[s]	579365	936508
	Core	8694[c]	250[f]	33[h]	100[i]	60[m]	60[q]	298[r]	343[s]	1175173	1352631
106	Long Span	9329[b]	50[d]	14[g]	100[i]	65[m]	70[n]	81.5[r]	134[s]	344872	567029
Tenant[a]	Short Span	4431[b]	42[e]	14[g]	100[i]	65[m]	82.5[n]	76.625[r]	138.5[s]	154006	278367
	Two Way	17497[b]	42[e]	16[g]	100[i]	65[m]	65[q]	74.25[r]	123[s]	589286	976190
	Core	8694[c]	250[f]	33[h]	100[i]	65[m]	65[q]	299.25[r]	348[s]	1180102	1372349
107	Long Span	9329[b]	50[d]	14[g]	100[i]	70[m]	70[n]	81.5[r]	134[s]	344872	567029
Rest.[a]	Short Span	4431[b]	42[e]	14[g]	100[i]	70[m]	82.5[n]	76.625[r]	138.5[s]	154006	278367
	Two Way	17497[b]	42[e]	16[g]	100[i]	70[m]	70[q]	75.5[r]	128[s]	599206	1015873
	Core	8694[c]	250[f]	33[h]	100[i]	70[m]	70[q]	300.5[r]	353[s]	1185031	1392067
108	Long Span	9329[b]	50[d]	141[j]	75[i]	75[m]	75[p]	209.75[r]	266[s]	887570	1125596
Lower	Short Span	4431[b]	42[e]	141[j]	75[i]	75[m]	75[p]	201.75[r]	258[s]	405491	518546
Mech.[a]	Two Way	17497[b]	42[e]	141[j]	75[i]	75[m]	75[p]	201.75[r]	258[s]	1601190	2047618
	Core	8694[c]	250[f]	141[k]	75[i]	75[m]	75[p]	409.75[r]	466[s]	1615862	1837686
109	Long Span	9329[b]	50[d]	141[j]	150[l]	80[m]	120[p]	221[r]	311[s]	935175	1316016
Upper	Short Span	4431[b]	42[e]	141[j]	150[l]	80[m]	120[p]	213[r]	303[s]	428102	608990
Mech.[a]	Two Way	17497[b]	42[e]	141[j]	150[l]	80[m]	120[p]	213[r]	303[s]	1690476	2404761
	Core	8694[c]	250[f]	141[k]	150[l]	80[m]	120[p]	421[r]	511[s]	1660227	2015145
110	Long Span	9329[b]	50[d]	14[g]	100[i]	85[m]	85[q]	85.25[r]	149[s]	360741	630503
Storage[a]	Short Span	4431[b]	42[e]	14[g]	100[i]	85[m]	85[q]	77.25[r]	141[s]	155262	283391
	Two Way	17497[b]	42[e]	16[g]	100[i]	85[m]	85[q]	79.25[r]	143[s]	628968	1134920
	Core	8694[c]	250[f]	33[h]	100[i]	85[m]	85[q]	304.25[r]	368[s]	1199820	1451220
Roof[a]	-	39951[c]	-	-	40[i]	100[m]	40[q]	10[r]	40[s]	181215	724859

[a] Table G-1 page 192 NIST-NCSTAR 1-2A
[b] Calculated from floor plan Figure 3-1 NIST-NCSTAR 1-2a page 29 reduced by 2.8259653% to match Table 3-1 NIST-NCSTAR 1-2B page 53
[c] Table 3-1 NIST-NCSTAR 1-2B page 53
[d] Figure 2-4 NIST-NCSTAR 1-1A page 11
[e] Figure 2-7 NIST-NCSTAR 1-1A page 19
[f] Figure 2-3 NIST-NCSTAR 1-1A page 10
[g] Table 4-1 NIST-NCSTAR 1-2A page 69
[h] Table 6-2 NIST-NCSTAR 1-2A page 137
[i] Table 5-1 NIST-NCSTAR 1-1 page 65
[j] Section 6.3.1 NIST-NCSTAR 1-2A page 140
[k] Table 6-6 NIST-NCSTAR 1-2A page 141
[l] ‘Hat Truss Floors’ NIST-NCSTAR 1-2A page 73
[m] Figure 2-9 NIST-NCSTAR 1-1A page 22
Reduced live load is highest value obtainable from either: [n] Figure 6-1 NIST-NCSTAR 1-2A page 136; [o] Table 5-1 NIST-NCSTAR 1-1 page 65; [p] Figure 2-8 NIST-NCSTAR 1-1a page 20; [q] Live Load Reduction as per Figure 2-9 NIST-NCSTAR 1-1A page 22
[r] CDL + SDL + 25% of reduced LL as per NIST-NCSTAR 1-2 page 67
[s] CDL + SDL + reduced LL
CDL = Construction Dead Load, SDL = Superimposed Dead Load, LL = Live Load

Appendix B – Stiffness calculation
i) sample of spreadsheet – full spreadsheet here:
http://www.filefactory.com/file/cd7fda5/n/Spreadsheet.xlsx

Column Number	area (sq. in)	area (sq. m)	floors	Up To Floor	height (m)	Stiffness individual beam	Stiffness of structure up to current floor	area (sq. in)	area (sq. m)	floors	Up To Floor	height (m)
501	893.5	0.5765	3	SL3	10.973	10.822	10.822	893.5	0.5765	3	1	10.973
502	612.75	0.3953	3	SL3	10.973	7.4217	7.4217	588.25	0.3795	3	1	10.973
503	570.94	0.3683	3	SL3	10.973	6.9152	6.9152	545.88	0.3522	3	1	10.973
504	402.25	0.2595	3	SL3	10.973	4.8721	4.8721	386.66	0.2495	3	1	10.973
505	426.94	0.2754	3	SL3	10.973	5.1711	5.1711	412.75	0.2663	3	1	10.973
506	640	0.4129	3	SL3	10.973	7.7517	7.7517	598.94	0.3864	3	1	10.973
507	598.94	0.3864	3	SL3	10.973	7.2544	7.2544	574.25	0.3705	3	1	10.973
508	893.5	0.5765	3	SL3	10.973	10.822	10.822	893.5	0.5765	3	1	10.973
601	360.69	0.2327	3	SL3	10.973	4.3687	4.3687	339.19	0.2188	3	1	10.973
602	305.44	0.1971	3	SL3	10.973	3.6995	3.6995	285.94	0.1845	3	1	10.973
603	285.94	0.1845	3	SL3	10.973	3.4633	3.4633	276	0.1781	3	1	10.973
604	214.94	0.1387	3	SL3	10.973	2.6033	2.6033	204.75	0.1321	3	1	10.973
605	229.98	0.1484	3	SL3	10.973	2.7856	2.7856	219.98	0.1419	3	1	10.973
606	285.94	0.1845	3	SL3	10.973	3.4633	3.4633	276	0.1781	3	1	10.973
607	293.25	0.1892	3	SL3	10.973	3.5519	3.5519	278.5	0.1797	3	1	10.973
608	317.19	0.2046	3	SL3	10.973	3.8418	3.8418	294.69	0.1901	3	1	10.973
701	369.75	0.2385	3	SL3	10.973	4.4784	4.4784	342.94	0.2212	3	1	10.973
702	343	0.2213	3	SL3	10.973	4.1544	4.1544	321.75	0.2076	3	1	10.973
703	226.38	0.146	3	SL3	10.973	2.7419	2.7419	219.98	0.1419	3	1	10.973
704	146.38	0.0944	3	SL3	10.973	1.7729	1.7729	133.27	0.086	3	1	10.973
705	106.25	0.0685	3	SL3	10.973	1.2869	1.2869	96.188	0.0621	3	1	10.973
706	248.75	0.1605	3	SL3	10.973	3.0129	3.0129	232.73	0.1502	3	1	10.973
707	337.25	0.2176	3	SL3	10.973	4.0848	4.0848	310.94	0.2006	3	1	10.973
708	362.63	0.234	3	SL3	10.973	4.3921	4.3921	301.25	0.1944	3	1	10.973
801	360	0.2323	3	SL3	10.973	4.3603	4.3603	336.19	0.2169	3	1	10.973
802	337.25	0.2176	3	SL3	10.973	4.0848	4.0848	310.94	0.2006	3	1	10.973
803	234.48	0.1513	3	SL3	10.973	2.8401	2.8401	224.86	0.1451	3	1	10.973
804	156.34	0.1009	3	SL3	10.973	1.8936	1.8936	146.94	0.0948	3	1	10.973
805	219.69	0.1417	3	SL3	10.973	2.6609	2.6609	203.98	0.1316	3	1	10.973

ii) Notes on Stiffness Data

General Notes

- Additional structural elements, such as diagonal bracing in the core and horizontal attachments for the perimeter columns, serve to reduce weakness at the joints between column sections and allow them to function as essentially vertical springs, as was modelled.

Perimeter Column Notes

- Perimeter columns were modelled as 12 individual corner columns and 57 wall sections consisting of a single beam from sublevel 6 to floor 9 then 3 beams from floor 9 to floor 107 (to match blueprints).
- NISTs diagrams show 59 columns on each side and 1 on each corner, this is contradicted by blueprints found on http://911research.wtc7.net/wtc/evidence/plans/table.html, more columns favours progressive collapse in this instance as it will increase stiffness so 240 perimeter columns was assumed
- NIST-NCSTAR 1-3 page 9 - perimeter columns thickness ranged from 0.25" to 3"
- http://911research.wtc7.net/mirrors/guardian2/wtc/godfrey.htm - wall thickness in perimeter columns decreased from 12.5mm to 7.5mm
- http://911research.wtc7.net/wtc/arch/perimeter.html - 3 diagrams showing varies thicknesses of wall in perimeter columns
- http://911research.wtc7.net/wtc/evidence/plans/table.html - clearly shows perimeter columns down to sublevel 6
- To match the various descriptions the following assumptions were made:
  - between sublevel 6 and floor 8 flange thickness ranged from 3" to 2.5", outer web ranged from 1" to 0.875" and inner web of 1"
  - from floor 9 to floor 107 the flange and web thickness ranged from 0.5" to 0.25" in 0.125" steps, NIST-NCSTAR 1-3 page 9 description of "Perimeter columns in the upper stories were...most commonly 0.25 in." was assumed to mean at least half the height and so 25% of the columns were 0.5", 25% 0.375" and 50% 0.25"
  - Above floor 107 no calculations were done, it is not relevant to the results as we are using stiffness from lower floors

Core Column Notes

- Column Cross Sections used with thanks from http://wtcmodel.wikidot.com/
- Data for column cross section extracted from SAP2000 data obtained under FOIA by Lon Waters of http://wtcmodel.wikidot.com/
- Cells highlighted yellow (except column 901) were incomplete cross sections, cross section showed only extra plates, correct beam was added from list in 'Selected WF I Beams'
- Column 901 had missing cross sections due to discrepency between Figure 2-6 (NIST NCSTAR 1-3 page 10) and blueprints on http://911research.wtc7.net/wtc/evidence/plans/index.html, however column 901 and 908 were an almost perfect match so correct beams were selected on this basis
- A sense check reveals that column cross sections match exactly descriptions given in 2.3.2 NIST NCSTAR 1-3 page 10 and Figure 2-7 page 11, there were some differences between transitions between box and WF beams as described in Figure 2-6 page 10 but they matched the blueprints and axial stiffness relies only of cross section area and length, not shape

A zip file with column cross sections can be found here :
http://www.filefactory.com/file/cd6c5c6/n/Core_Column_Cross_Sections.rar

A copy of the spreadsheet can be found here: http://www.filefactory.com/file/cd7fda5/n/Spreadsheet.xlsx