Progressive Collapse and Bridge Dynamics

Uwe Starossek, Prof., P.E., Hamburg University of Technology, Hamburg, Germany

Abstract

An account is given on recent research activities at the Structural Analysis and Steel Structures Institute of Hamburg University of Technology related to progressive collapse and bridge dynamics. Progressive collapse is characterized by a distinct disproportion between a triggering event and the resulting widespread collapse. A general typology of progressive collapse is introduced and examples are given of the occurrence of such events and how they can be prevented in bridges. Furthermore, a general design approach to preventing progressive collapse based on a decision-making process and deterministic structural analysis is presented. As a second topic, the determination of instationary wind force coefficients for bridge girders, i.e., the so-called flutter derivatives, is treated. Flutter derivatives for nine different cross sections were measured in model tests in a water channel. The flutter derivatives were also determined numerically for the same nine, and many other, cross sections by using computational fluid dynamics. The raw and processed data of all tests and numerical calculations, as well as a comparison between both, have been posted online and are publicly available. Finally, two devices for vibration control developed and being investigated at the Institute are presented. The so-called aeroelastic damper consists of a small tuned mass damper inside the bridge girder that is connected via a mechanical transmission to aerodynamic control surfaces on the outside. It is a passive device that does not depend on external energy supply or electric and electronic circuitry and thus is highly reliable. The basic unit of the so-called twin rotor damper consists of two rotating unbalanced masses that are combined in various and varying configurations. The resulting control force is a directed oscillating force, an oscillating moment, or a combination of both. In comparison to conventional active vibration control devices, it has the advantage of a drastic reduction of energy demand and actuator power and size. Wind-tunnel tests impressively demonstrate the effectiveness of the device. Videos of the tests have been posted online at www.tuhh.de/sdb.

1 Introduction

Topics of recent research carried out at the Structural Analysis and Steel Structures Institute of Hamburg University of Technology range from progressive collapse of structures over wind-induced vibrations and vibration control to the structural behavior of concrete-filled steel tubes and the design and analysis of integral bridges. An account is given here on the three topics that relate to the theme of this Seminar on Dynamics and Progressive Collapse in Cable-Stayed Bridges: progressive collapse, wind-induced vibrations, and vibration control.

2 Progressive collapse

Progressive collapse is characterized by a distinct disproportion between cause and effect, i.e., between a triggering event and the resulting widespread collapse. The triggering event can be a local action or a local lack of resistance that leads to an initial local failure. Such failure can remain limited or it can lead to a collapse progression and cause major damage. When the failure remains limited, the structure is called robust, otherwise it is called non-robust. Normal design procedures do not consider initial local failure and, therefore, do not distinguish between robust and non-robust structures. A uniform level of safety can thus not be achieved, at least not without additional considerations concerning a possible collapse progression after an initial local failure [1, 2].

2.1 Typology of progressive collapse

Although the disproportion between cause and effect is a defining and common feature of progressive collapse, there are various differing mechanisms of collapse that produce such an outcome. The amenability to conceptual, theoretical, and computational treatment can vary accordingly. Collapse-promoting fea-
tures, possible or preferable countermeasures, and the suitability of indices for quantifying robustness and collapse resistance will likewise depend on the mechanism of collapse. Finally, different kinds of structures are susceptible to different mechanisms of collapse. It is thus useful to distinguish and describe the different types of progressive collapse and to attempt a classification on that basis. Six different types of collapse and, at a higher level of abstraction, four classes of collapse have been identified so far [2, 3]. Two of the six types are presented in the following: pancake-type collapse and zipper-type collapse.

**Pancake-type collapse** is exemplified by the collapse of the World Trade Center towers on 11 September 2001. The impact of the aircrafts and the subsequent fires led to damage and resulted in local failures in the areas of aircraft impact. The ensuing loss in vertical load-carrying capacity was limited to a few storeys but extended over the entire cross section of the respective tower. The upper part of the structure started to fall and to accumulate kinetic energy. The subsequent collision with the lower part, which was still intact, caused large impact forces that were far beyond the reserve capacities of the structure. This structural impact thus again led to the loss of vertical load-carrying capacity over the entire cross section of the tower in the area of impact. Failure progressed in the same manner and led to total collapse. The term suggested for this type of collapse originates from the appearance of smaller buildings after collapse (Fig. 1). Their potential energy is smaller and hence a smaller degree at destruction occurs when compared to the WTC towers. The floor slabs of the building pile on top of each other like in a stack of pancakes.

![Fig. 1. Pancake-type collapse of a ten-storey building triggered by an earthquake (Islamabad, 2005)](image)

**Zipper-type collapse** is exemplified by the collapse of the Tacoma Narrows Bridge triggered by wind-induced vibrations (1940).

![Fig. 2. Zipper-type collapse of the Tacoma Narrows Bridge triggered by wind-induced vibrations (1940)](image)
The mechanism of a pancake-type collapse exhibits the following characteristic features [2, 3]:

- initial failure of vertical load-bearing elements,
- separation of structural components and their fall in vertical rigid-body motion,
- transformation of gravitational potential energy into kinetic energy,
- impact of separated and falling components on the remaining structure,
- failure of other vertical load-bearing elements due to impact-induced axial compression,
- failure progression in the vertical direction.

Characteristic features are the separation of structural components, the release of gravitational energy, and, in particular, the occurrence of impact forces. The gravitational potential energy released during the fall can far exceed the elastic potential energy (strain energy) stored in the structure. If that energy is reintroduced into the structure in a subsequent impact, large internal forces ensue. Due to the dynamic nature of impact, they tend to concentrate in the load-bearing elements directly affected by the impact. The failure progresses when these forces cause the impacted elements to fail as well. Their actual failure can be associated with any local failure mode including buckling.

Zipper-type collapse can be observed in the footage of the collapse of the original Tacoma Narrows Bridge in 1940 (Fig. 2). After the first hangers snapped due to excessive wind-induced distortion of the bridge girder, the entire girder peeled off from the hangers and suspension cables and fell. A similar mechanism of collapse can be envisaged in cable-stayed bridges or in anchored retaining walls, where progressive collapse could be initiated by the failure of one or a few cables or anchors. The UK Standing Committee on Structural Safety (SCOSS) has drawn attention to a number of collapses of heavy acoustic ceilings in cinemas and other buildings which seem to correspond to zipper-type collapse triggered by the failure of one or a few inadequate fixings to the supporting structure [4]. In the examples just given, zipper-type collapse is related to the initial failure of tension elements. This, however, is not a necessary condition for this type of collapse, which follows from a description of characteristic features.

The mechanism of a zipper-type collapse exhibits the following characteristic features [2, 3]:

- initial failure of one or a few load-bearing elements,
- redistribution of the static forces carried by these elements into the remaining structure,
- impulsive dynamic loading due to the suddenness of the initial failure and redistribution of forces,
- dynamic response of the remaining structure to that impulsive dynamic loading,
- concentration of forces in other load-bearing due to the combined static and dynamic effects,
- overloading and failure of these elements,
- failure progression in a direction transverse to the forces in the failing elements.

Characteristic features are, in particular, the redistribution of forces into alternative load paths, but also an impulsive loading due to sudden element failure and a concentration of static and dynamic forces in the next elements to fail. Impact forces do not typically occur – in contrast to the pancake-type collapse discussed before. Again, element failure can be associated with any local failure mode including buckling.

2.2 Progressive collapse of bridges and their prevention

The Viadotto Cannavino, Italy, collapsed during construction in 1972 [5]. The triggering event was a form-work failure. The ensuing collapse was due to lack of robustness of the structural system. Another disproportionate collapse occurred during the construction of Haeng-Ju Grand Bridge, Seoul in 1992 [2]. After the failure of one span of that continuous prestressed concrete girder bridge, the collapse progressed through the adjacent ten spans, and an 800 m section of bridge was lost. In both cases, the continuous prestressing tendons in the superstructure of the bridge played a particular and disastrous role. When the Haeng-Ju Grand Bridge collapsed, most tendons resisted the enormous stresses caused by the rupture of the encasing concrete and the failure and fall of structural elements. The forces transmitted by the continuous prestressing tendons thus enabled the collapse to progress. Remarkably, it came to a halt at expansion joints at both ends of the 800 m section. A different course of events took place during the failure of Tasman Bridge near Hobart, Australia, in 1975 [6, 7]. Due to a ship impact, two piers of the bridge were demolished and three spans of deck supported by them collapsed. The other 19 spans remained in-
tact. The bridge deck was constructed of precast prestressed concrete beams. The non-occurrence of collapse progression and, thus, the robustness of that bridge were related to the discontinuity of prestressing tendons between adjacent spans, which effectively led to a segmentation of the structure. The collapse of the I-35W Mississippi River Bridge in Minneapolis, on the other hand, has partly been ascribed to the absence of alternative load paths within the structural system of that bridge [8]. Because alternative paths require continuity, this case seems to indicate that the robustness of a system can also increase with its degree of continuity, contrary to the previous examples.

The segmentation approach was used in the design of the Confederation Bridge, Canada, a prestressed concrete bridge that consists of 43 continuous main spans of 250 m each and shorter approach spans (Fig. 3). If this structure were to be made robust and collapse resistant by alternative load paths, an initial local failure to be assumed would be the failure of a bridge pier. This in turn would require the design of a prestressed concrete frame with a double span length of 500 m – arguably a vain endeavor. Thus, the design method chosen was to limit the local failure by isolating the collapse through a segmentation of the structure [2, 9]. In consultation with the supervising authority the acceptable extent of collapse was determined and, based thereupon, the location of collapse boundaries was derived. The collapse must not transgress these boundaries; the collapsing part of the structure is thus isolated from the remaining structure. The collapse boundaries are structurally implemented through properly designed hinge corbels within the bridge girder at which the failed portion of the girder can safely disengage from the remaining structure.

In other types of bridges, the alternative-path approach is more appropriate. In the design of cable-stayed bridges, it is usually required to consider the sudden loss, for what reason whatsoever, of any one cable. It must be demonstrated that the structure safely endures such event. Because the force of a ruptured cable must be redistributed to the remaining structure and the other cables, this design approach is equivalent to providing alternative paths. The Post-Tensioning Institute (PTI) recommends that the impulsive dynamic loading resulting from the sudden loss of a cable is to be determined in a quasi-static analysis using a dynamic amplification factor (DAF) of 2.0. Application of such recommendation in the design of recently erected cable-stayed bridges in the U.S. showed that the loss-of-cable load case can become a controlling requirement for the design of the bridge girder increasing construction costs. For this reason, the recommendation that a DAF determined by non-linear dynamic analysis, but not smaller than 1.5, can be used as an alternative was added to the 5th edition of the PTI Recommendations [10].

To gain more information about appropriate values of the DAF, numerical studies were performed for a cable-stayed bridge with a main span of 600 m (Fig. 4) [2, 11-13]. The two cable planes consist of 80 ca-
bles each. The cable anchors in the bridge girder are spaced at 15 m. The latter consists of an orthotropic deck supported on two steel edge beams with a height of 2.60 m. The effect of loss-of-cable load cases is investigated in non-linear dynamic analyses of a spatial model of the structure. The loss of any one cable is modeled as a sudden failure and the ensuing extreme responses of state variables (displacements, sectional forces) are determined. By relating these extreme dynamic responses to the corresponding static responses to cable loss, DAFs are derived. Because the comparison of dynamic and static responses is done case by case for individual cable losses and state variables, the resulting DAF is valid for the considered case only and will generally be different for other cable losses and state variables.

![Fig. 4. Structural system for analysis of loss-of-cable load cases](image)

The following range of results for the DAFs was obtained for the given state variables:

**Vertical deflection of bridge deck**
- downwards: \(1.5 - 1.8\)
- upwards: \(1.55 - 4\) (and higher)

**Bending moments in bridge deck**
- positive: \(1.3 - 1.6\)
- negative: \(1.4 - 2.7\)

**Normal forces in bridge deck**
- long cables: \(1.9 - 2.3\)
- short cables: \(> 2.0\)

**Cable forces:** \(1.35 - 2.0\)

Note that for some of the above state variables DAFs higher than 2.0 can occur. This is even more so the case for the bending moments in the pylons. The corresponding DAFs were found to be in the range of 10 to 30, i.e., one order of magnitude larger than 2.0. These findings indicate that, at least for the pylon moments due to sudden cable loss, a quasi-static analysis with a DAF of 2.0 is unsafe and should be replaced by proper dynamic analysis.

### 2.3 General design approach to preventing progressive collapse

A general design approach has been developed for preventing progressive collapse (Fig. 5) \[2, 14, 15\]. It consists of design requirements and design objectives, to be defined by client and authorities, and design methods and verification procedures, to be selected and performed by the engineer. The design objectives include hazard scenarios and performance objectives. The available design methods are event control, protection, increased local resistance, alternative load paths, and segmentation. The prediction of the structural response to local failure – needed for applying the design methods alternative load paths and segmentation – requires suitable verification procedures. It is suggested to use deterministic structural analysis, which should be as accurate as possible and consider all relevant scenarios. In general, non-linear dynamic analysis using multi-degree-of-freedom models is required. Note that the design approach outlined here has been adopted by the Disproportionate Collapse Standards and Guidance (DCSG) Committee of the Structural Engineering Institute (SEI) of the American Society of Civil Engineers (ASCE) as the basis of the guidelines to be developed by the Committee.
Fig. 5. General design approach to preventing progressive collapse

- design requirements
  - how necessary is design against disproportionate collapse?
    - exposure
    - significance
  - classification
    - no/basic/high/very high requirement level
  - design recommendations
    - no req.: no additional assessment
    - basic req.: indirect design, direct design
    - high req.: direct design (non-threat-specific)
    - very high req.: direct design (threat-specific and non-threat-specific)

- design objectives
  - hazard scenarios
    - abnormal design conditions
    - threat-specific: specific abnormal events
    - non-threat-specific: notional actions, notional damage
  - performance objectives
    - global: acceptable extent of collapse, acceptable other damage
    - local: acceptable rotation of plastic hinges
  - combinations of actions and safety factors

- design methods
  - event control
  - protection
  - increased local resistance
  - alternative load paths
  - segmentation
  → depending on
    - type of structure
    - requirement level
    - design objectives

- verification procedures
  - are performance objectives met when hazard scenarios occur?
    - analytical verification
      - detailed analysis: non-linear dynamic analysis using MDOF models
      - simplified analysis: linear analysis, quasi-static analysis, generalized SDOF models
    - model or full-scale tests
  - (in indirect design, verification is replaced by checking the proper application of prescriptive design rules)
3 Numerical and experimental determination of flutter derivatives

An extensive research program has been performed to determine the instationary wind force coefficients, i.e., the so-called flutter derivatives, for various kinds of bridge girder cross sections [16, 17]. Flutter derivatives for nine different cross sections, including those of Great Belt Bridge, Millau Viaduct, Severn Bridge, Chongquing Bridge, and the original Tacoma Narrows Bridge, were measured through forced-vibration sectional model tests in a water channel. The flutter derivatives were also determined numerically for the same nine, and many other, cross sections by simulating forced vibrations of the respective profiles in an two-dimensional air flow by using computational fluid dynamics.

\[
\begin{align*}
\mathbf{L} &= \rho \pi b^2 \begin{pmatrix} c_{hh} & b c_{ha} \\ b c_{dh} & b^2 c_{aa} \end{pmatrix} \\
\text{Reduced frequency:} \\
k &= \omega b / v
\end{align*}
\]

Cross sections: (Great Belt, Denmark); (Chongquing, China)

Fig. 6. Real and imaginary parts of experimentally and numerically determined flutter derivatives

Comparison between experimentally and numerically determined flutter derivatives generally shows good agreement for plate-like cross sections (Great Belt, Millau, Severn, Chongquing; see Fig. 6) and poor agreement for non-plate-like cross sections (Tacoma). The raw and processed data of all tests and numerical calculations, as well as a comparison between both, have been posted online and are publicly available [18].

4 Vibration control devices

Various devices for the control of structural vibrations have been developed and are being investigated at our institute. Two of them are presented here: the aeroelastic damper and the twin rotor damper.

4.1 Aeroelastic damper

The aeroelastic damper consists of a small tuned mass damper (TMD) inside the bridge girder that is connected via a mechanical transmission to an aerodynamic control surface on the outside (Fig. 7) [19]. Once the bridge starts to vibrate, the tuned mass damper will move relative to the bridge girder. This motion is translated via the mechanical transmission into angular displacements of the aerodynamic control surface. This in turn changes the flow field around the entire bridge girder inducing aerodynamic control forces that dampen the motion of the bridge.
The aeroelastic damper has been patented. It is a passive control device that does not depend on external energy supply or electric and electronic circuitry and, therefore, is highly reliable. It is being investigated at our institute to verify its effectiveness and to establish guidelines for choosing optimum design parameters. The latter include the mechanical parameters of TMD and transmission, the position of the control surfaces relative to the bridge girder, and the position of their pivot points within the control surfaces. These problems are investigated analytically on the basis of the equations of motion, experimentally in the wind tunnel, and numerically by using computational fluid dynamics [19].

4.2 Twin rotor damper

The basic unit of the twin rotor damper consists of two rotating unbalanced masses that are combined in various and varying configurations [20, 21]. The resulting control force is a directed oscillating force, an oscillating moment, or a combination of both. In the configuration exemplified in Fig. 8, two twin rotor dampers are combined. In each of both devices, two masses rotate with the same constant angular speed in opposite directions around a longitudinal axis. Both masses of one device point in the upward direction when the masses of the other device point in the downward direction, and vice versa. The resultant of the generated centrifugal forces of each device is a harmonic vertical force. The phase difference between the resultant forces of both devices is 180º. The overall resultant effect thus is a harmonic moment around the longitudinal axis of the bridge girder. This moment can be used for the control of torsional vibrations due to, e.g., buffeting or flutter.

The twin rotor damper has been patented. It is an active control device that needs energy supply, actuators (i.e., motors), sensors to measure the structural motion to be damped, and a control algorithm implemented through electric and electronic hardware and software. In comparison to conventional active control devices, however, it has the advantage of a drastic reduction of energy demand and actuator power and size. This is due to a virtually constant angular speed of the control masses. Only small accelerations are required for fine-tuning the motion of the device to the motion of the structure – contrary to conventional active control devices, in which the control masses perform linear motions and, therefore, must constantly be accelerated or decelerated.

The device is applicable to a large variety of structures and structural vibration problems including vibration-induced impairment of structural stability (e.g., wind-induced flutter vibration of long-span bridges). Because of the reliability concern often associated with active devices, however, its main field of applica-
tion might rather be the mitigation of serviceability and fatigue problems (e.g., pedestrian-induced vibrations of footbridges, operation-induced vibrations of cranes and wind turbine towers).

The twin rotor damper is being investigated at our institute to verify its effectiveness, to develop control strategies and algorithms for various kinds of application, and to establish guidelines for choosing optimum design parameters [20, 21]. These problems are investigated analytically on the basis of the equations of motion, numerically by means of Matlab simulations, and experimentally in free-vibration and wind tunnel tests. Recent sectional model tests in our wind tunnel impressively demonstrate the device’s effectiveness for controlling flutter vibration of bridges. Videos of these and other tests have been posted online at www.tuhh.de/sdb [22].

Conclusion

An account has been given on research activities at the Structural Analysis and Steel Structures Institute of Hamburg University of Technology that relate to the theme of this Seminar on Dynamics and Progressive Collapse in Cable-Stayed Bridges: progressive collapse, wind-induced vibrations, and vibration control. A common denominator of all these activities is that they had originally been inspired by the author’s own professional experience in designing long-span bridges. Interestingly, results have been obtained that are valid for, and applicable to, not only bridges but the entire realm of engineering structures.

References


[18] Starossek, U. Flutter derivatives for various sections obtained from experiments and numerical simulations (data files). Structural Analysis and Steel Structures Institute, Hamburg University of Technology, 2009, available online at www.tuhh.de/sdb.


